ADDIS ABABA SCIENCE AND TECHNOLOG UNIVERSITY

COMPARATIVE STUDY OF CONCENTRIC AND ECCENTRIC BRACING SYSTEM FOR LATERAL LOADS ON HIGH RISE IRREGULAR STEEL BUILDING

A thesis submitted to the School of Graduate Studies in Partial fulfillment of the Requirements for the Degree of Master of Science in Structural Engineering

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Declaration of Authorship

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work. This research work is original and has not been submitted for the award of any other Master Degree, either in this or any other.

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ABSTRACT

The resistance to the lateral loads from wind or from an earthquake is the reason for the evolution of various structural systems. Because, when a high-rise or any multi-level structure is subjected to lateral or tortional deflections under the action of seismic loads; the resulting oscillatory movement can induce a wide range of responses in the building. As a result, lateral stiffness is a major consideration in the design of tall buildings. In addition to this, many existing steel buildings and reinforced concrete buildings for which the poor lateral stiffness is the main problem; need to be retrofitted to overcome the deficiencies to resist the lateral loading. Lateral load resisting systems are structural elements providing basic lateral strength and stiffness, without which the structure would be laterally unstable. The unstable nature of structure is solved by appropriate provision of bracings systems.

Bracing system is a structural system which forms an integral part of the frame. Thus, such a structure has to be analyzed before arriving at the best type or effective arrangement of bracing. Bracing is a highly efficient and economical method of resisting lateral forces in a frame structure because the diagonals work in axial stress and therefore call for minimum member sizes in providing the stiffness and strength against horizontal shear.

Different literature review indicate that there is enough research on braced frame but mostly it is either experimental study or Finite element analysis of single bay regular two storey frame. Some macro model studies have been also done but limited to five to fifteen storey 2D frame steel building. So in this study, Earthquake analysis is done on G+25 steel building making T plan with 3D modeling (i.e. high rise framed building) in seismic zone IV (Adama City) for seismic ,dead and live loads to see the effect on both conditions i.e. with and without different bracing.

Bracing structures are widely utilized in steel buildings to increase the resistance of the overall structural systems. But the resistance capacities of bracings are different for different orientation of bracing systems. Previous studies said that X bracing performs better than any other concentrically bracing type. But their criteria of measurement are not stipulated clearly. To minimize such set back, this study considers the weight of the bracing assumed to be a constant parameter for all selected bracing type. The building has been modeled and analyzed using ETABS software making 15 horizontal bays of width 4 meters, and storey height of 4m due to lateral earthquake as per Ethiopian Building Code of standard.

The performance of the same steel building has been investigated for different types of bracing system such as concentric (crossed X) bracing, combination of V and inverted V or chevron bracing, diagonal bracing, knee bracing and four eccentric bracing types using channel sections as bracing and I – sections for beams and columns.

Depending on the analysis result the stability of the building has been evaluated in terms of lateral story displacement and storey drift at different story level. The effectiveness of the above types of steel bracings to the building has also been investigated. More importantly, the reduction in lateral displacement has been found out for different types of bracing system in comparison to building with no bracing. In this study, both concentric (single diagonal, alternate direction bracing) and eccentric V type bracing greatly reduce lateral displacement and thus significantly contributes to high stiffness to the structure. Whereas even though eccentric v type brace reduce more lateral displacement, it is not as economical as single diagonal, alternate direction bracing. From both concentric and eccentric bracing systems studied; concentric single diagonal, alternate direction bracing arranged as diamond shape is the best economically as well as in providing lateral stiffness to the structure.

Keywords: Bracing system, concentric and eccentric bracing, lateral storey displacement,

Storey drifty

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ABBREVIATIONS AND NOTATIONS

AISC- American institute of steel construction

ASCE- American society of civil engineering

CBFs - Concentrically braced frames

CP -Collapse Prevention

EBFs- Eccentrically braced frames

EBCS- Ethiopian building code of standards

EN- European standard

ETABS- Extended Three-Dimensional Analysis of Building Systems

IO-Immediate Occupancy,

LFRS- lateral force resisting systems

LL- Live load

LS- Life Safety

MRFs- Moment resisting frames

NA- National annex

SCBF-Special concentrically braced frames

ULS- Ultimate limit state

SLS- Serviceability limit state

F_y- Nominal yield strength

f_u- Ultimate tensile strength

H- Height of the building from basement

K- Effective length factor

q- Structural behavior factor

R-Response modification factor

T1-Fundamental period of vibration

RCC – Reinforced concrete column

DCM – Medium ductility class

DCH – High ductility class

SDOF – Single degree of freedom

MDOF- Multi degree of freedom

CHAPTER 1. INTRODUCTION, OBJECTIVE AND SIGNIFICANCE OF THE STUDY

1.1. Introduction

When a tall building is subjected to lateral or torsion deflections under the action of fluctuating seismic loads; the resulting oscillatory movement can induce a wide range of responses in the building's occupants from mild discomfort to acute nausea. As far as the ultimate limit state is concerned, lateral deflections must be limited to prevent second order p-delta effect due to gravity loading being of such a magnitude which may be sufficient to precipitate collapse. To satisfy strength and serviceability limit states, lateral stiffness is a major consideration in the design of tall buildings. The simple parameter that is used to estimate the lateral stiffness of a building is the drift index defined as the ratio of the maximum deflections at the top of the building to the total height of the building. Different structural forms of tall buildings can be used to improve the lateral stiffness and to reduce the drift index. In this research, study is conducted for braced frame structure in which bracing is a highly efficient and economical method to laterally stiffen the frame structures against wind load and earth quake load. The efficiency of the bracing is due to the diagonals work in axial stress and therefore calls for minimum member sizes in providing the stiffness and strength against horizontal shear. Thus it is an important priority for a good structural design engineer to select the best and economical bracing system for the high rise steel structures.

A bracing system is a structural system which is designed primarily to resist seismic forces. Steel bracing is one of such system which is economical, easy to erect, occupies less space and has flexibility to design for meeting the required strength and stiffness. It is usually placed in vertically aligned spans. This system allows obtaining a great increase of stiffness with a minimal added weight, and so it is very effective to use in design of steel structure and for existing structure for which the poor lateral stiffness is the main problem. Bracings are usually provided to increase stiffness and stability of the structure under lateral loading and also to reduce lateral displacement significantly. They are designed to work in tension and compression similar to a truss. They virtually eliminate the columns and girder bending factors and thus improve the efficiency of the pure rigid frame actions. By the addition of truss members such as diagonals (between the floor systems) this can be achieved effectively. These diagonals carry lateral loads and transfers the axial loads to the columns, which is an effective structural system. There are mainly two types of bracing systems.

1. Concentric bracing system.
2. Eccentric bracing system.

1. Concentric bracings - These are the type of bracings whose centroidal axis coincides with each other. They mainly increase the lateral stiffness of the frame which in turn increases the natural frequency and also decreases the lateral storey drift. The reason why concentric bracing increase the natural frequency of the building is that natural period (T_n) of a building is inversely proportional to the stiffness of the building. Thus, when concentric bracing increase the stiffness of the frame, natural period of the building decrease which in turn increases natural frequency due to natural frequency is the reciprocal of natural period of the building. Mathematically:

$T_n = 2\pi\sqrt{m/k}$ and $f_n = 1/T_n$

Where: T_n = natural period, f_n = natural frequency of the building, m = mass of the building, k = stiffness

Further, the bracing increases the axial compression in the columns to which they are connected by decreasing the bending moments and shear forces in the column.

2. Eccentric bracings - These are the type of bracings whose centerline braces are offset from the intersection of the centerline of columns and beams. They mainly improve the energy dissipation capacity and reduce the lateral stiffness of the system. Due to eccentric connection of the braces to beams, the lateral stiffness of the system depends upon the flexural stiffness of the beams. At the point of connection of eccentric bracings to the beams, the vertical component of the bracing force due to earthquake exists. This vertical component of the bracing force due to earthquake causes concentrated lateral load on the beams at the point of connection of the eccentric bracings. Eccentrically braced frames can be used as this have a well-established reputation as high-ductility systems and have the potential to offer cost-effective solutions in moderate seismic region.

This paper explores the structural behavior of steel building for both bracing system and un braced conditions under static (dead and live loads on the building) and lateral loading. The results of non linear static analysis have been presented and discussed in this paper. Finally, a comparative study has been presented to assess the best structural performance of steel building under lateral loading. The main aim of my research work has been to identify the type of bracing which causes minimum storey displacement such contributes to greater lateral stiffness to the structure.

1.2 Background of the Study

While there are no universally accepted definitions for the standard height of the buildings Bureau of Planning and Sustainability of Addis Ababa city proposes the distinction as:

Low rise = 1-6 stories, medium = 7- 12 stories, high rise = 13 and above. Structural systems of these buildings need resistance mechanism especially in areas of high seismic regions to sustain its stability without sudden collapse. Hence bracing structures are the most widely utilized in steel buildings to increase the resistance of the overall structural systems. But the resistance capacities of bracings are different for different orientation of bracing systems. Previous studies from International Journal of Science and Research said that X bracing performs better than any other concentrically bracing type). But their criteria of measurement are not stipulated clearly. To minimize such set back, this study considers the weight of the bracing assumed to be a constant parameter for all selected bracing type.

Thus to compare the efficiencies of bracings; four types concentric and four types of eccentric steel bracings are selected in this research. They are:

- X bracing for one storey
- Combination of V and inverted V bracing which forms X bracing for two storey
- Diagonal bracing system (single diagonal, alternate direction bracing)
- Knee bracing system (one member is connected to the midpoint of the other)
- V-bracings (eccentric)
- Eccentric bracings (three types)

✓ Each of these concentric and eccentric bracing types is provided to twenty-five storied, T-shape irregular steel building. Then this building is modeled and analyzed using ETABs Nonlinear version 9.7.1 which is finite element based soft ware.

1.3 Objective and Significance of the Study

1.3.1 General objective

The general objective of this thesis is to compare and evaluate the effectiveness of concentric and eccentric bracing systems on high rise irregular steel building structure under lateral loads due to seismic load.

1.3.2 Specific objectives

a) To identify the bracing, which causes minimum Storey displacement from both concentric and eccentric bracing systems.
b) To identify the efficient and economic bracing system to laterally stiffen the frame structures against seismic load.
c) To compare various parametric results such as Storey drift, Storey displacement, maximum bending and axial forces induced in the frames for both types of bracing systems.

1.3.3 Significance of the Study

The scope of the study is to select the most efficient; seismic load resistant bracing type which gives the minimum lateral displacement out of the types of bracings assumed and to compare the effectiveness of concentric and eccentric bracings.

The advantageous of outcomes of the study are:

- Consulting firms can benefit from the output of this research work.
- It will increase awareness of practicing architects and structural engineers about Configuration of concentric and eccentric bracing systems for high rise building
- It will thus avoid arbitrarily locating types of steel brace in steel buildings.
- It is an important priority for a good structural design engineer to select the best and economical bracing system for the high rise steel structures.

1.4. Content of Thesis

The study considered both eccentrically and concentrically type of bracing systems having a structural resistance capacity for lateral loads through a vertical concentric and eccentric truss systems. The axes of the members are made to align concentrically at the joints in case of concentric bracing system and centerlines of braces are offset from the intersection of the centerline of beam column joints in case of eccentric brace.

This study is limited to X bracing, combination of V and inverted V or chevron bracing, diagonal bracing, knee bracing, V bracing, and other three types of eccentric bracings, except v bracing which is eccentric type. During comparison; the study did not consider any aesthetical effects of the bracing for the provision of doors and windows.

The study depends on twenty-five storied T shape irregular a steel building which is analyzed with the provision of different bracing types such as; X- bracing, combination of v and inverted V-bracing (chevron bracing), diagonal bracing, knee bracing, V-bracing and other three eccentric bracing types. The general classification of these bracing types; based on their geometrical arrangements are selected from concentric bracings and eccentric bracings.
The storey height and bay width of the building is assumed to be equal to 4m, for equal treatment of bracing which do not alter the behavior of bracings. In addition to this, the weight of each type of bracings is assumed to be equal which is constant parameter in this work. For analysis of this steel building, Euro code 3- Design of steel structures; and Euro code 8- Design of structures for earthquake resistance, are used. These codes have direct similarity to that of the new EBCS 3 and EBCS 8 of 2013 version.

The content of this thesis is organized in different sections which are arranged as follows:

a) Section one deals with an introductory part which include background, objective, Significance of the study and contents of the thesis.
b) Section two briefly reviews theoretical background of steel bracing systems, classifications, principles and design approaches are considered.
c) Section three discusses about the modeling software and loading consideration in the frame geometry is highlighted.
d) Section four tells about the analysis of structural systems for the given loading under consideration.
e) Section five presents comparison and discussion for lateral displacement and storey drift, for each of the bracing type investigated using Microsoft excel program with the help of graphs.
f) Finally, conclusions drawn and recommendation is forwarded to show research areas for the next researchers.

CHAPTER 2. LITRETURE REVIEW

2.1. Recent Research Work

E.M. Hines and C.C. Jacob [2009] presented a paper on Eccentric braced frame system performance. According to his paper the seismic performance of low-ductility steel systems designed for moderate seismic regions have generated new interest in the cost-effective design of ductile systems for such regions. Although eccentrically braced frames (EBFs) have a well-established reputation as high-ductility systems and have the potential to offer cost-effective solutions in moderate seismic regions, their system performance has not been widely discussed. Eccentrically Braced Frames (EBFs) are also known for their attractive combination of high elastic stiffness and superior inelastic performance characteristics (AISC 2005).

The University of California, Berkeley (UCB) under the direction of Professors Popov and Bertero also conducted a test of two separate 0.3 scale shake table tests of Concentrically Braced Frame (CBF) and EBF dual systems (Uang and Bertero 1986, Whittaker et al. 1987, Whittaker et al. 1990).

The design of shear links for the tower of the San Francisco-Oakland Bay Bridge East Bay selfanchored suspension span (McDaniel et al. 2003), studied on performance based plastic design of steel concentric braced frames for enhanced confidence level in China. Concentrically braced frames (CBFs) are generally considered less ductile seismic resistant structures than other systems due to the brace buckling or fracture when subjected to large cyclic displacements. This is attributed to simpler design and high efficiency of CBFs compared to other systems such as moment frames, especially after the 1994 Northridge Earthquake. However, recent analytical studies have shown that CBFs designed by conventional elastic design method suffered severe damage or even collapse. Conventional bracing systems include typical diagonal and chevron bracing configurations, as well as innovative concepts such as strut-to-ground and zipper braced frames (Khatib et al. 1988, Bruneau et al. 1998). Seismic regulations and guidelines for the seismic design of CBFs can be found in the Structural Engineers Association of California (SEAOC) Recommended Lateral Force Requirements (SEAOC 1996), the International Building Code (IBC 2000), the NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings (BSSC 2000), and the AISC Seismic Provisions for Structural Steel Buildings (AISC 2002).

Diagonal and chevron systems can provide large lateral strength and rigidity but do not provide great ductility as buckling of the diagonals leads to rapid loss of strength without much force redistribution (Goel, 1992). In chevron brace the unbalanced vertical forces that arise at the connections to the floor beams due to the unequal axial capacity of the braces in tension and compression causes deterioration of lateral strength of the frame. In order to prevent undesirable deterioration of lateral strength of the frame, the provisions require that the beam should possess adequate strength to resist this potentially significant post-buckling force redistribution.The adverse effect of the unbalanced vertical force at the beam-to-brace connections can be mitigated by adding zipper elements, as proposed by Khatib et al. (1988). If the compression brace in the first story buckles while all other braces remain elastic, a vertical unbalanced force is then applied at the middle span of the first story beam. The zipper elements mobilize the stiffness of all beams and remaining braces to resist this unbalance. The unbalanced force transmitted through the zipper elements increases the compression of the second story compression brace, eventually causing it to buckle.

P. Uriz and S.A. Mahin (2004) presented a paper on Seismic performance assessment of concentrically braced steel frames. The overall their investigation includes systems that utilize conventional braces, buckling restrained braces and braces incorporating viscous damping devices. In the first part the same reliability framework as used to assess Special Moment Resisting Frame (SMRF) structures during the FEMA/SAC Steel Project was employed to assess the confidence with which Special Concentric Braced Frames (SCBF) and Buckling Restrained Braced Frames (BRBF) might achieve the seismic performance expected of new SMRF construction. In the second part, a test program to improve modeling of SCBF systems was described, including the design of a nearly full-size, two-story SCBF test specimen. The confidence that a three story SCBF designed according to the 1997 NEHRP provisions was able to achieve the collapse prevention performance goal was less than 10% for all definitions capacity and a seismic hazard corresponding to a 2% probability of exceedance in 50 years. A similarly designed six-story BRBF was demonstrated to be much more reliable. The performance-based evaluation approach for characterizing and improving the performance of steel braced frames incorporating conventional bracing, buckling restrained braces, friction and hysteretic devices, and viscous dampers.

C.Y. Ho and G.G. Schierele [1990] published a journal paper on Effect of configuration and lateral drift on High-rise space frames. According to his paper Excessive lateral drift of high-rise frames can damage secondary systems, such as partitions walls; generate secondary column stress due to P-δ moments; and cause discomfort to building occupants under prolonged cyclical drift. Damage to secondary system can be controlled by reducing drift. However the P-δ effect is most severe in moment resisting frames; the Uniform Building Code allows smaller seismic drift for moment resisting frames (0.3% story drift vs. 0.5 % for other systems). Design for wind or seismic forces are usually based on objectives to minimize lateral drift.

To reduce lateral drift of high-rise building is an important design consideration in areas of high wind and/or seismic activity. The research presented here shows that selecting the most appropriate bracing system can substantially reduce drift with only minor cost differences.

2.2. Structural type and Behavioral factor of Steel Structures

Steel is a versatile construction material widely used in the construction of high rise structures, bridges, airport hangers, shopping complex, rope car pylons, recreational structures, steel arch, etc. It has high strength and ductility, which is the primary requirement under seismic action because the structure has to absorb the vibration energy imparted to it during shaking of ground. Thus, steel buildings are more flexible than RCC buildings, but also they display more lateral displacement than RCC buildings which can be controlled by providing lateral support mechanism like bracing structures. Structural planning of steel buildings should conform to that the beams yield prior to the columns, and the strength of a connection should be greater than the strength of beams and columns framing into the connection members and connections should guarantee high strength, ductility, and energy dissipation capacity, and an excessive lateral sway should be avoided.

Multi-storey buildings are generally constructed in steel as framed structures. A ductile frame can undergo important inelastic deformations, localized in the neighborhood of sections with maximum bending moment. This eventually leads to the formation and rotation of plastic hinges and redistribution of plastic moments, allowing the structure to resist higher loads than those predicted by the elastic analysis. Un-braced steel buildings are ductile and possess large energy dissipation capacity but tend to deform greatly, causing serious damage to non-structural elements during small to medium-size earthquakes. Braced frames can resist large amounts of lateral forces and have reduced lateral deflection and thus reduced P-Δ effect. However, a uniform distribution of bracing throughout the structure is desirable.

2.2.1. Structural Type

Steel buildings shall be assigned to one of the following types according to their behavior under seismic action.

a) Moment resisting frame, which resists horizontal forces acting in an essentially flexural manner. In these structures the dissipative zones are mainly located in plastic hinge near the beam -column joints and energy is dissipated by means of cyclic bending.

b) Concentric braced frames, in which the horizontal forces are mainly resisted by members subjected to axial forces. In this structure the dissipative zones are mainly located in the tensile diagonals. Concentric braced frames can be divided into the following categories.

   (i) Active tension diagonal bracing, in which the horizontal forces can be resisted by tension diagonals only, neglecting compression diagonals.

   (ii) V–Bracing, in which horizontal forces can be resisted by considering both tension and compression diagonals. The intersection points of these diagonals lie on horizontal member which must be continuous.

   (iii) K–bracing, in which the diagonal intersection ties on column. This category must not be considered as dissipative when the yielding mechanism involves the yielding of the column.

c) Eccentric braced frames, in which the horizontal forces are mainly resisted by axial loaded members and the eccentricity of the layout such that energy can be dissipated in the beams by means of either cyclic bending or shearing. Eccentricity braced frames can only be classified as dissipative due to bending or shearing the bending members precedes the attainment of the limit strengths of the tension and compression members.

d) Cantilever structures or inverted pendulum structures, as defined in clause 4 of EBCS 8 and in which dissipative zones are mainly located at the base.

e) Structures with concert cores or concrete walls, in which the horizontal forces are mainly 24resisted by these cores or walls.

f) Dual structures as defined in clause 4.1.2 of EBCS.

2.2.2 Behavior factors (γ)

1. The behavior factor γ introduced in 1.4.2.4 of EBCS 8 to account for energy dissipative takes the value provided for regularity requirement.
2. If the building is not regular in elevation the γ value should be increased by 20% (but need not taken more than γ =1)
3. For regular buildings in zone 1 and 2 having structural system made from rolled sections or from welded sections with similar size as rolled sections confirming to the available structural types, a behavior factor γ =0.7 may be adopted.

2.3. Causes and Failure Modes of Steel Structures

Although steel is highly ductile, inelastic ductility is necessarily retained in the finished structure. Hence, care must be taken during design and construction to avoid losing this property. Considerable care is also needed to check failures due to instability and brittle fracture to the development of full ductility and energy dissipation capacity under earthquake loading.

The causes of instability are:

(i) **Local buckling of plate elements** (e.g., web, flange, etc.) with large width to-thickness ratios: A steel member containing plate elements with a large width-to thickness ratio is unable to reach its yield strength, because of prior local buckling. Even if the yield strength is attained, ductility will be inadequate. Under cyclic loading, it is observed that strength and ductility decrease with increasing width-to-thickness ratio, and local buckling of web causes further degradation.

(ii) **Flexural buckling of long columns and braces**: Long columns may fail by buckling. This mode of instability is sudden and can occur when the axial load in a column may never reach the yield. Even a small lateral force in such condition will produce a substantial deflection leading to instability and the phenomenon is called flexural buckling. The capacity of slender columns is, therefore, limited by the stiffness of the member rather than the strength of the material. Thus, the lateral stiffness of the frames is increased by bracing the frames. However, buckling of braces is a potential source of instability of steel frames. Steel bracing dissipate considerable energy by yielding under tension but buckle without much energy dissipation in compression. Therefore, the energy dissipation capacity of concentrically braced frames is marked less, due to buckling of braces than that of the moment frames.

(iii) **Lateral-tensional buckling of beams**: During moderate to strong shaking of the ground, additional forces are developed in various members of a structure. For a beam loaded in flexure, the load bearing side (generally the top) carries the load in compression, whereas the non-load bearing side (generally the bottom) will be in tension. If the beam is not supported in the opposite direction of bending, and the flexural load increases to a critical limit, the beam will fail due to local buckling on the compression side in wide-flange sections designed for flexure only. If the top flange buckles laterally, the rest of the section will twist resulting in a failure mode known as lateral-torsional buckling.

(iv) **P-Δ effects**: in frames subjected to large vertical loads: If the lateral stiffness is inadequately high, the building as a whole, or one or more stories, can fail due to the P-Δ effect. This is because of the secondary effect on shears and moments of the frame members, due to the action of the vertical loads, which interact with the lateral displacement of the building resulting from seismic forces.

(v) **Uplift of braced frames**: Earthquakes have a vertical component of movement in addition to the traditionally considered horizontal effects. The stresses produced due to vertical motion are generally considered not to be significant to cause instability. However, due to the horizontal component of movement, the overturning moments produce additional longitudinal stresses in walls and columns and additional upward (uplifting) and downward (thrust) forces in foundations causing instability.

(vi) **Connection failure**: The failures of bolted and welded connections are to be avoided.

The causes of brittle failure in steel buildings are that brittle failure is more frequent in welded steel structures, particularly, those that are fillet welded, than it is in structures connected by mechanical fasteners. This is due to a combination of possible weld defects, high residual stresses, stress concentration, which reduce the possibility of crack arrest, tension failure at net sections of bolted or riveted connection, and Lamellar tearing of plates in which the through-thickness strain due to weld metal shrinkage is large and highly restrained.

It is evident that the main objectives to achieve adequate performance of steel buildings are: the use of sufficiently ductile steel, and the ductile design and fabrication of framed members and connections. All frame instability, especially the excessive sway leading to higher levels of damage to non-structural components and to higher secondary stresses due to P-Δ effect, should be avoided; all forms of brittle failures should be avoided; and also failure mechanism should provide maximum redundancy, i.e., the possibility of failure by local collapse should be avoided. All portions of the building should be tied well together.

2.4. Lateral Load Resisting Systems

Lateral load resisting systems are structural elements that provide its basic lateral strength and stiffness, and without which the structure would be laterally unstable.

The resistance of tall buildings to wind as well as to earthquake is the main determinant in the formulation of new structural systems that evolve by the continuous efforts of structural engineers to increase building height while keeping the deflection within acceptable limits and minimizing the amount of materials.

Thanks to the sophisticated computer technology, modern materials and innovative structural concepts, structural systems have gone beyond the traditional frame construction of the home insurance building and have allowed skyscrapers to grow to the greater heights now a day.

Most of the tallest buildings in the world have steel structural system, due to its high strength-to-weight ratio, ease of assembly and installation, economy in transport to the site, availability of various strength levels, and wider selection of sections. Innovative framing systems and modern design methods, improved fire protection, corrosion resistance, fabrication, and erection techniques combined with the advanced analytical techniques made possible by computers, have also permitted the use of steel in just any rational structural system for tall buildings.

Buildings are basically big cantilever beams which are supported on one end only and the loads are perpendicular to the beam. As in a beam, buildings are designed for strength (shear and flexure) and serviceability (deflection).

Structural engineering of tall buildings requires the use of different systems for different building heights. Each system, therefore, has an economical height range, beyond which a different system is required. The requirements of these systems and their ranges are somewhat imprecise because the demands imposed on the structure significantly influence these systems. However, knowledge of different structural systems, their approximate ranges of application, and the premium that would result in extending their range is indispensable for a successful solution of a tall building project.

2.5. Moment Resisting-Rigid Frame Systems

Moment frames develop their resistance to lateral forces through the flexural strength and continuity of beam and column elements. They are utilized in both steel and reinforced concrete construction. Rigid frame systems for resisting lateral and vertical loads have long been accepted for the design of the buildings. Rigid framing, namely moment framing, is based on the fact that beam-to-column connections have enough rigidity to hold the nearly unchanged original angles between intersecting components. Owing to the natural monolithically behavior, hence the inherent stiffness of the joist, rigid framing is ideally suitable for reinforced concrete buildings. On the other hand, for steel buildings, rigid framing is done by modifying the joints by increasing the stiffness in order to maintain enough rigidity in the joints.

The fundamental requirements for all ductile moment frames are that:

i. They have sufficient strength to resist seismic demands,

ii. They have sufficient stiffness to limit inter-story drift,

iii. Beam-column joints have the ductility to sustain the rotations they are subjected to,

iv. Elements can form plastic hinges, and

v. Beams will develop hinges before the columns at locations distributed throughout the structure.

For a rigid frame, the strength and stiffness are proportional to the dimension of the beam and the column dimension, and inversely proportional to the column spacing. Columns are placed where they are least disturbing to the architecture, but at spacing close enough to allow a minimum depth of floor. Thus, in order to obtain an efficient frame action, closely spaced columns and deep beams at the building exterior must be used. Especially for the buildings constructed in seismic zones, special attention should be given to the design and detailing of joints, since rigid frames are more ductile and less vulnerable to severe earthquakes when compared to steel- braced.

2.6. Building Irregularities

The impact of irregularities in estimating seismic force levels, first introduced into the Uniform Building Code in 1973, long remained a matter of engineering judgment, beginning in 1988, however, some configuration parameters have been quantified to establish the condition of irregularity, and specific analytical treatments have been mandated to address these conditions.

Typical building configuration deficiencies include an irregular geometry, a weakness in a story, a concentration of mass, or a discontinuity in the lateral-force-resisting system. Although these are evaluated separately, they are related and may occur simultaneously. The 1997 UBC quantifies the idea of irregularity by defining geometrically or by use of dimensional ratios, the points at which the specific irregularity becomes an issue requiring remedial measures. It should be noted that not all irregularities require remedial measures. Some, such as stiffness, mass, and geometric irregularities, may be accounted for by performing dynamic analysis. A building with an irregular configuration may be designed to meet all code requirements, but it will not perform as a building with a regular configuration. If the building has an odd shape that is not properly considered in the design, good details and construction are of a secondary value.

The irregularities are divided into two broad categories:

1) Vertical; and

2) Plan irregularities.

Vertical irregularities include soft or weak stories, large changes in mass from floor to floor, and large discontinuities in the dimensions or in-plane locations of lateral-load-resisting elements. Buildings with plan irregularities include those that undergo substantial torsion when subjected to seismic loads or have reentrant corners, discontinuities in floor diaphragms, discontinuity in the lateral force path, or lateral-load resisting elements that are not parallel to each other or to the axes of the building. These irregularities result in building responses significantly different from those assumed in the equivalent static force procedure. Although most codes give certain recommendations for assessing the degree of irregularity and corresponding penalties and restrictions, it is important to understand that these recommendations are not an endorsement of their design; rather, the intent is to make the designer aware of the potential detrimental effects of irregularities. If the configuration of a building has an inside corner, then it is considered to have a reentrant corner. It is the characteristic of buildings with an L, H, T, X, or variations of these shapes. Two problems related to seismic performance are created by these shapes:

1) Differential Vibrations between different wings of the building may result in a local stress concentration at the reentrant corner; and

2) Torsion may result because the center of rigidity and center of mass for this configuration do not coincide.

There are two alternative solutions to this problem: Tie the building together at lines of stress concentration and locate seismic-resisting elements at the extremity of the wings to reduce torsion, or separate the building into simple shapes. If the building is separated; the width of the separation joint must allow for the estimated inelastic deflections of adjacent wings. The purpose of the separation is to allow adjoining portions of buildings to respond to earthquake ground motions independently without pounding on each other. If it is decided to dispense with the separation joints, collectors at the intersection must be added to transfer forces across the intersection areas. Since the free ends of the wings tend to distort most, it is beneficial to place seismic-resisting members at these locations.

According to Universal Building Code (UBC) 1997, both vertical and plan irregularities are sub-divided and explained as the following.

2.6.1. Plan irregularities

2.6.1.1. Torsional irregularity-to be considered when diaphragms are not flexible

Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure. In this case increase torsion forces by an amplification factor *Ax*.

2.6.1.2. Reentrant corners

Plan configurations of a structure and its lateral force- resisting system contain reentrant corners, Where both projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction. Provide structural elements in diaphragms to resist flapping actions.

2.6.1.3. Diaphragm discontinuity

Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed area of the diaphragm, or changes ineffective diaphragm stiffness of more than 50% from one story to the next. Thus, structural elements have to be provided to transfer forces into the diaphragm and structural system and boundaries at openings are reinforced.

2.6.1.4. Out-of-plane offsets

Discontinuities in a lateral-force path, such as out-of-plane offsets of the vertical elements. Use special seismic load combinations. One-third increase in stress is not permitted. Therefore braced frames have to be designed as per UBC (Uniform Building Code) 2213.8.

2.6.1.5. Nonparallel systems

The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system. This is designed for orthogonal effects.

2.7. Dynamic Analysis of Irregular building.

Symmetrical buildings with uniform mass and stiffness distribution behave in a fairly predictable manner, whereas buildings that are asymmetrical or with areas of discontinuity or irregularity do not. For such buildings, dynamic analysis is used to determine significant response characteristics such as:

1) The effects of the structure's dynamic characteristics on the vertical distribution of lateral forces.

2) The increase in dynamic loads due to tensional motions; and

3) The influence of higher modes, resulting in an increase in story shears and deformations.

Static methods specified in building codes are based on single-mode response with simple corrections for including higher mode effects. While appropriate for simple regular structures, the simplified procedures do not take into account the full range of seismic behavior of complex structures. Therefore, dynamic analysis is the preferred method for the design of buildings with unusual or irregular geometry.

Two methods of dynamic analysis are permitted:

1) Elastic response spectrum analysis and

2) Elastic or inelastic time-history analysis.

The response spectrum analysis is the preferred method because it is easier to use. The time-history procedure is used if it is important to represent inelastic response characteristics or to incorporate time-dependent effects when computing the structure's dynamic response.

Structures that are built into the ground and extended vertically some distance above ground respond as either simple or complex oscillators when subjected to seismic ground motions. Simple oscillators are represented by single-degree-of-freedom systems (SDOF), and complex oscillators are represented by multidegree-of-freedom (MDOF) systems.

2.8. Bracing systems

2.8.1. Introduction

Bracing resists horizontal forces such as wind, crane longitudinal surge, and earthquake load. Every fourth or fifth bay may be braced. But no less than two should be provided. The type of bracing can be single diagonal members or cross members, v brace, inverted v brace, knee brace, K brace, and mirror of k brace, L brace and others. Single bracing members must be designed to carry loads in tension and compression. With cross-bracing, only the members in tension are assumed to be effective and those in compression are ignored. In addition to bracings, the internal frames resist the transverse wind load by bending in the cantilever columns.

2.8.2. Braced Frame Systems

Braced frame systems are mostly utilized in steel buildings since the diagonal bracing has used to resist tension for one or the other directions of lateral loading. Concrete bracing of the double diagonal form is sometimes used, however, with each diagonal designed as a compression member to carry the full external shear. Contrary to rigid frame, having less elastic stiffness and low energy dissipation capacity, this system is a highly efficient and economical for resisting horizontal loading and attempts to improve the effectiveness of a rigid frame by almost eliminating the bending of columns and girders, by the help of additional bracings. It behaves structurally like a vertical truss, and comprises of the usual columns and girders, essentially carrying the gravity loads, and diagonal bracing components so that the total set of members forms a vertical cantilever truss to resist the horizontal loading.

Bracing generally takes the form of steel rolled sections, circular bar sections, or tubes. The areas around elevator, stairs, and service shafts, where frame diagonals may be enclosed within permanent walls, are the most preferable places for the braces; and the arrangement of the bracing is generally dictated by the requirements for openings. They can cover two or more than two stories in a single run which gives high strength and ductility of the structure with number of stories. This configuration is well suited for tall, slender buildings and was firstly used in a steel building, the 100-storey-high John Hancock Center (1969).

Historically, bracing has been utilized to stabilize the building laterally in many of the world's tallest structures, including 77-storey-high Chrysler Building (1930) and 102-storey-high Empire State Building (1931) in New York.

The outcome of an earthquake manifests great devastation due to unpredicted seismic motion striking extensive damage to innumerable buildings of varying degree, i.e. either full or partial. This damage to structures in turn causes irreparable loss of life with a large number of casualties. Strengthening of structures using bracing systems proves to be a better option. A bracing system improves the seismic performance of the frame by increasing its lateral stiffness and capacity. Through the addition of the bracing system, load could be transferred out of the frame and into the braces, bypassing the weak columns while increasing strength. In braced frames the lateral resistance of the structure is provided by diagonal members that, together with the girders, form the "web" of the vertical truss, with the columns acting as the "chords". Because the horizontal shear on the building is resisted by the horizontal components of the axial tensile or compressive actions in the web members. Bracing systems are highly efficient in resisting lateral loads. As per EBCS 3 definition; a frame may be classified as braced if its sway resistance is supplied by a bracing system with a response to in-plane horizontal loads which is sufficiently stiff for it to be acceptably accurate to assume that all horizontal loads are resisted by the bracing system. This may be assumed to be the case if the frame attracts not more than 10% of the horizontal loads.

The efficiency of bracing, is being able to produce a laterally very stiff structure for a minimum of additional material, makes it an economical structural form for any height of building, up to the very tallest. An additional advantage of full triangulated bracing is that the girders usually participate only minimally in the lateral bracing action: consequently, the floor framing design is independent of its level in the structure and, therefore, can be repetitive up the height of the building with obvious economy in design and fabrication. A major disadvantage of diagonal bracing is that it obstructs the internal planning and the location of windows and doors. For this reason, braced bents are usually incorporated internally along wall and partition lines, and especially around elevator, stair, and service shafts. Another drawback is that the diagonal connections are expensive to fabricate and erect.

Steel braced frame is one of the structural systems used to resist earthquake loads in multistoried buildings. It is an economical, easy to erect, occupies less space and has flexibility to design for meeting the required strength and stiffness. It is a highly efficient and economical method of resisting horizontal forces in a frame structure. Thus it has been used to stabilize laterally the majority of the world’s tallest building structures. It is also efficient because the diagonals work in axial stress and therefore call for minimum member sizes in providing stiffness and strength against horizontal shear. It has immense advantages not only in high rise structures but also in single story steel buildings of industrial buildings, airplane hangars or warehouse buildings. For the lateral load resisting systems for such tall structures, design engineers often use vertical braced frames with two or more bracing tiers or panels stacked between the ground and roof level with this configuration, braced length is reduced, which leads to smaller brace size as shorter brace are more effective in compression. The application of seismic system using braces is one of the most effective methods in steel structures. The most important issues in the study of this kind of systems are to determine the appropriate types of bracing.

Lateral resistance in braced frames is provided by diagonal members which forms the vertical truss structure together with the main beams. Columns in this structure are basic members. Since the shear forces are supported by horizontal components of tensile or compressive axial forces, bracing systems are very efficient. The desired behavior of bracing system in generation of lateral stiffness with minimum amount of materials, reveal it as an economic solution for a variety of buildings with arbitrary height. Another advantage of diagonal bracings is that the main beams have minimum participation in resisting of lateral loads and therefore of deck systems in different stories can be designed in a repetitive manner that is more desirable in economical point of view.

In braced frames, the primary source of drift capacity is through buckling and yielding of diagonal brace members. Proportioning and detailing rules for braces ensure adequate axial ductility, which translates into lateral drift capacity for the system.

Special design and detailing rules for connections, beams and columns attempt to preclude less ductile modes of response that might result in reduced lateral drift capacity.

2.8.3. Types of Bracings

Today braces in the constructions play a major role in supporting and integrating the whole structures of the buildings which minimizes the failure cases of structures. Furthermore, various types of braces embrace different strength of force. In a multi-storey building, the beams and columns are generally arranged in an orthogonal pattern in both elevation and on plan. In a braced frame building, the resistance to horizontal forces is provided by two orthogonal bracing systems.

2.8.3.1 Horizontal bracing:

At each floor level, bracing in a horizontal plane, generally provided by floor plate action, provides a load path to transfer the horizontal forces (mainly from the perimeter columns, due to wind pressure on the cladding) to the planes of vertical bracing. A horizontal bracing system is needed at each floor level, to transfer horizontal forces (chiefly the forces transferred from the perimeter columns) to the planes of vertical bracing that provide resistance to horizontal forces. There are two types of horizontal bracing system that are used in multi-storey braced frames.

- Diaphragms
- Discrete triangulated bracing.

2.8.3.1.1 Horizontal Diaphragms

Usually, the floor system will be sufficient to act as a diaphragm without the need for additional steel bracing. At roof level, bracing, often known as a wind girder, may be required to carry the horizontal forces at the top of the columns, if there is no diaphragm.

All floor solutions involving permanent formwork such as metal decking fixed by through-deck stud welding to the beams, with in-situ concrete infill, provide an excellent rigid diaphragm to carry horizontal forces to the bracing system. And also, floor systems involving precast concrete planks require proper consideration to ensure adequate transfer of forces if they are to act as a diaphragm. The coefficient of friction between planks and steelwork may be as low as 0.1, and even lower if the steel is painted. This will allow the slabs to move relative to each other, and to slide over the steelwork. Grouting between the slabs will only partially overcome this problem, and for large shears, a more positive tying system will be required between the slabs and from the slabs to the steelwork. Connection between slabs may be achieved by reinforcement in the topping. This may be mesh, or ties may be placed along both ends of a set of planks to ensure the whole panel acts as one. Typically, a 10 mm bar at half depth of the topping will be satisfactory.

Connection to the steelwork may be achieved by one of two methods:

1. Enclose the slabs by a steel frame (on shelf angles, or specially provided constraint) and fill the gap with concrete.
2. Provide ties between the topping and an in-situ topping to the steelwork (known as an 'edge strip'). Provide the steel beam with some form of shear connectors to transfer forces between the in-situ edge strip and the steelwork.

If plan diaphragm forces are transferred to the steelwork via direct bearing (typically the slab may bear on the face of a column), the capacity of the connection should be checked. The capacity is generally limited by local crushing of the plank. In every case, the gap between the plank and the steel should be made good with in-situ concrete. However, timber floors and floors constructed from precast concreted inverted tee beams and infill blocks (often known as 'beam and pot' floors) are not considered to provide an adequate diaphragm without special measures.

2.8.3.1.2. Discrete Triangulated Bracing

Where diaphragm action from the floor cannot be relied upon, a horizontal system of triangulated steel bracing is recommended. A horizontal bracing system may need to be provided in each orthogonal direction. Typically, horizontal bracing systems span between the 'supports', which are the locations of the vertical bracing. This arrangement often leads to a truss spanning the full width of the building, with a depth equal to the bay centers. This floor bracing is frequently arranged as a Warren truss, or as a Pratt truss, or with crossed members.

2.8.3.2. Vertical bracing:

Bracing in vertical planes (between lines of columns) provides load paths to transfer horizontal forces to ground level and provide a stiff resistance against overall sway. In a braced multi-storey building, the planes of vertical bracing are usually provided by diagonal bracing between two lines of columns. Either single diagonal is provided (in which case they must be designed for either tension or compression) or crossed diagonals are provided (in which case slender bracing members carrying only tension may be provided).

This system allows obtaining a great increase of stiffness with a minimal added weight, and so it is very effective for existing structure for which the poor lateral stiffness is the main problem.

Note that when crossed diagonals are used and it is assumed that only the tensile diagonals provide resistance, the floor beams participate as part of the bracing system (in effect a vertical Pratt truss is created, with diagonals in tension and posts in compression).

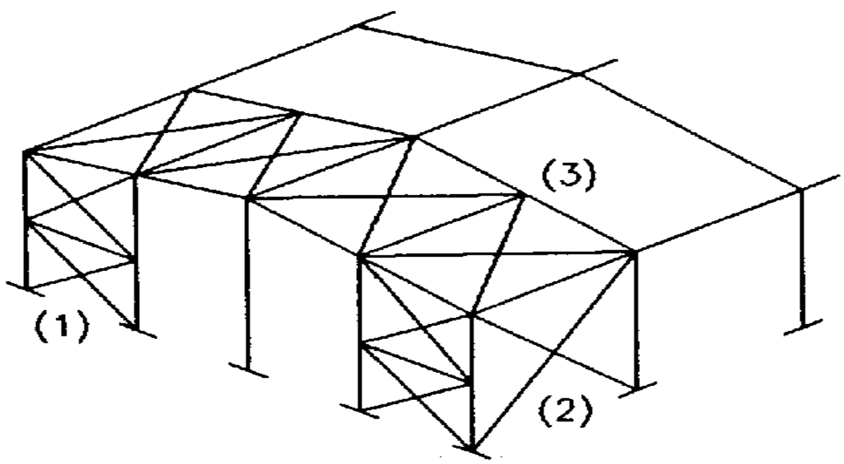
The vertical bracing must be designed to resist the forces due to the following:

- Wind loads
- Equivalent horizontal forces, representing the effect of initial imperfections
- Second order effects due to sway (if the frame is flexible).

Forces in the individual members of the bracing system must be determined for the appropriate combinations of actions. For bracing members, design forces at ULS due to the combination where wind load is the leading action are likely to be the most difficult ones. In this study; emphasis is given more on vertical bracing systems.

2.8.3.2.1. Classification of Vertical Bracings

Even though the shape and arrangements of bracings are various, based on its geometrical arrangements of the member, it can be classified as in to two types called concentrically bracing and eccentrically bracing system. Both types of bracings run diagonally from vertical member to the horizontal members (i.e. columns to beams) or from beam-column joint to other joint diagonally. This system allows obtaining a great increase of stiffness with a minimal added weight, and so it is very effective for existing structure for which the poor lateral stiffness is the main problem.



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1) Transverse vertical bracing
2) Longitudinal vertical bracing
3) Plan bracing

Figure 1.1- Transverse, Longitudinal and plan bracing

2.8.4 Eccentrically Braced Frames

These are the type of bracing whose centerline braces are offset from the intersection of the centerline of columns and beams. It mainly improves the energy dissipation capacity and reduces the lateral stiffness of the system. At the point of connection of eccentric bracings on the beams, the vertical component of the bracing force due to earthquake causes concentrated load on the beams. Eccentrically braced frames (EBFs) are a lateral load resisting systems for steel building that can be considered as hybrid between conventional moment–resisting frames (MRFs) and concentrically braced frames (CBFs). They are in effect an attempt to combine the individual advantages of MRFs and CBFs, while minimizing their respective disadvantages. Figure 1.2, 1.3, 1.4 and 1.5 bellows are several common EBF arrangements. The distinguishing characteristics of an EBF is that at least one end of every brace is connected so that the brace force is transmitted either to another brace or to a column through shear and bending in a beam segment called a link. The link length in figure 1.2 and 1.5 bellow is designated by the letter e.

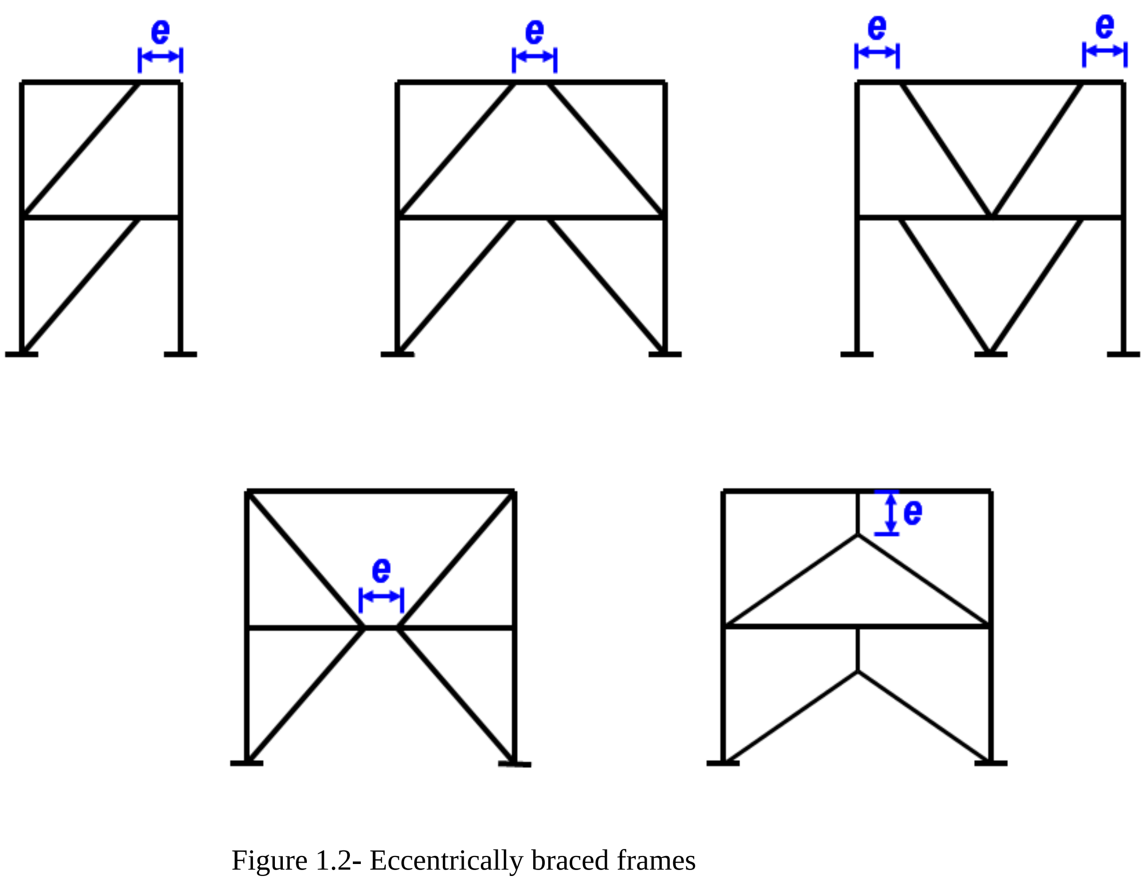


Figure 1.2- Eccentrically braced frames

Figure 1.3- Eccentrically braced building (AISC 2002)

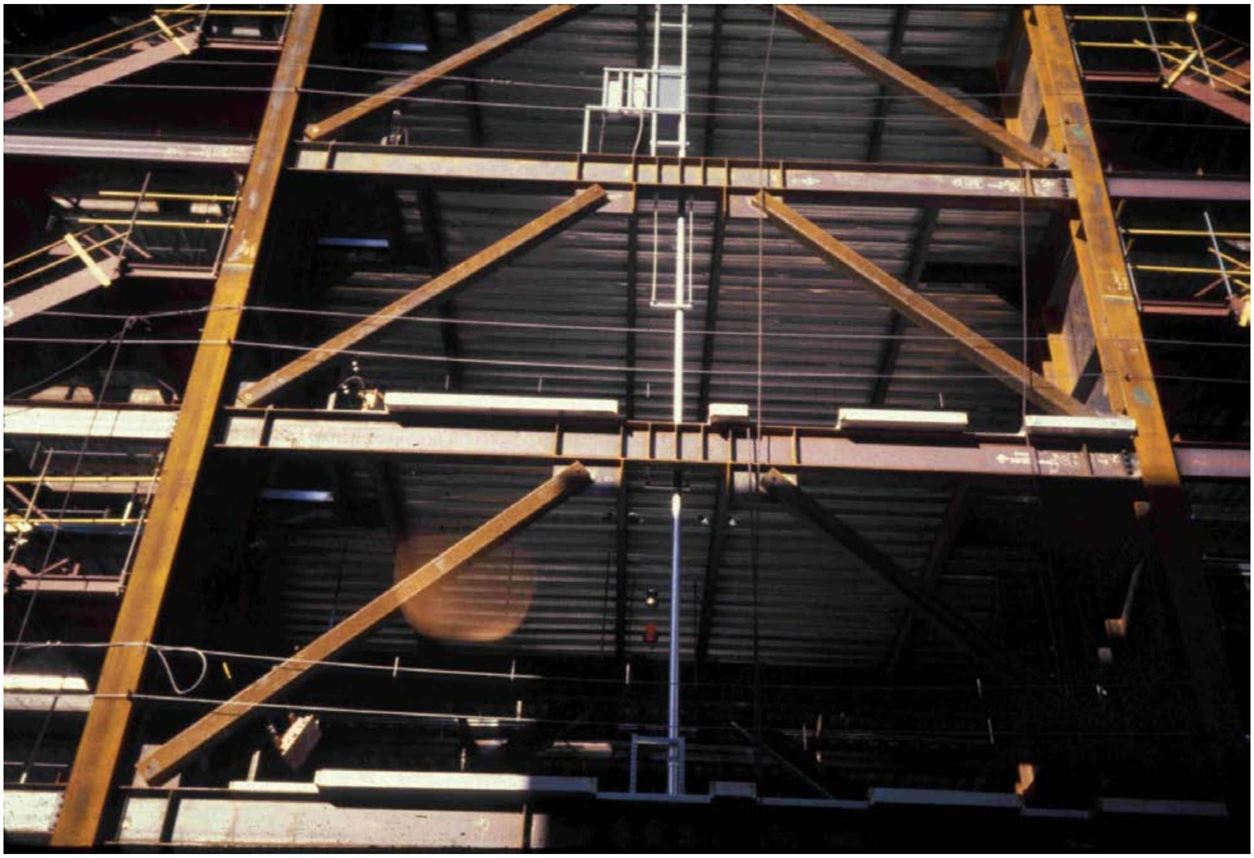


Figure 1.4- Eccentrically braced frames (AISC 2005)

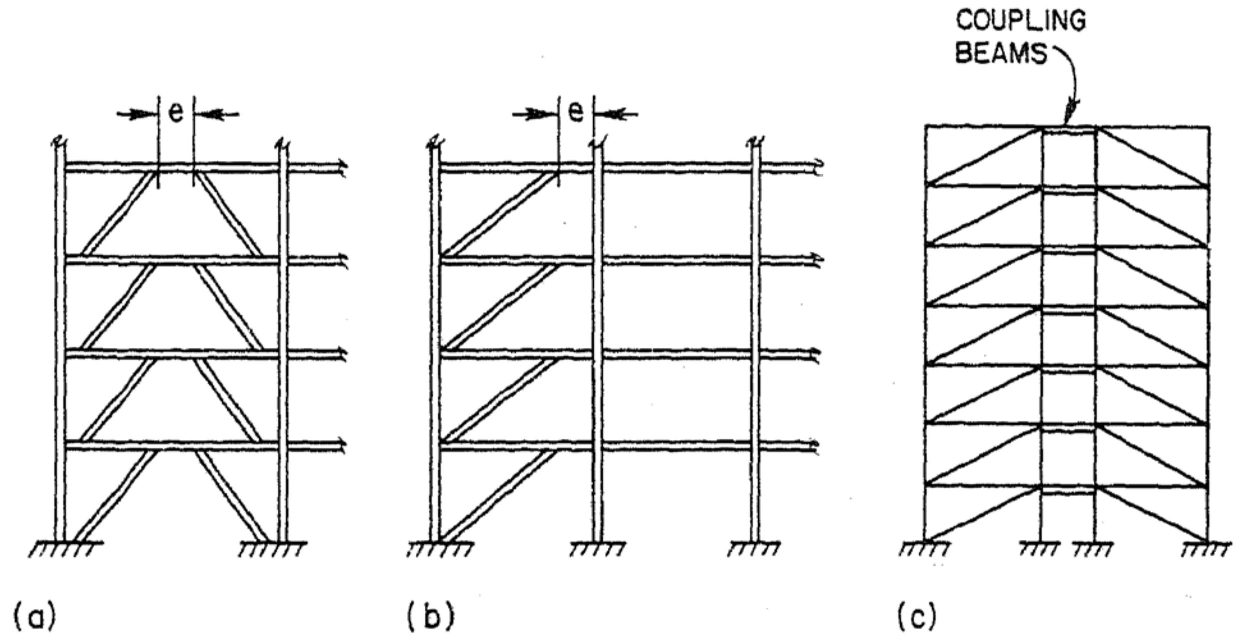


Figure 1.5- Eccentrically braced frames

Although eccentric bracing has been long known for wind bracing, its application to seismic resistant construction is only very resent. The excellent performance of EBFs under severe earthquake loading was demonstrated on one- third –scale model frames at the University of California in 1977. Soon after this study, several major buildings were constructed incorporating EBFs as part of their lateral seismic resisting systems, including the nineteen story Bank of America building in San Diego and the forty-seven story embarcadero four building in San Francisco , which is constructed in 1981. Since that time, numerous applications of these systems have been adopted in practice. Because eccentric bracings reduce the lateral stiffness of the system and improve the energy dissipation capacity. Due to eccentric connection of the braces to beams, the lateral stiffness of the system depends upon the flexural stiffness of the beams and columns, thus reducing the lateral stiffness of the frame. The most attractive features of EBFs for seismic-resistance design are their high stiffness combined with excellent ductility and energy-dissipation capacity. The bracing members in EBFs provide the high elastic stiffness which is characteristic of CBFs, permitting code drift requirements to be met economically.

Yet, under very severe earthquake loading, properly designed and detailed EBFs provide the ductility and energy dissipation capacity which is the characteristics of moment resisting frames (MRFs).The excellent ductility of EBFs can be attributed to two factors:

a) First, inelastic activity under severe cyclic loading is restricted primarily to the links, which are designed and detailed to sustain large inelastic deformations without loss of strength.

b) Secondly, braces are designed not to buckle, regardless of the severity of lateral loading on the frame.

The yielding of the links in EBFs serves to limit the maximum force transferred to the brace, acting, in effect, as a fuse for bracing member loads. The ultimate strength of the link can be accurately estimated. Thus, by designing the brace to be stronger than the link the designer can be assured with a high degree of confidence that the brace will not buckle, regardless of the severity of the earthquake load. The rapid deterioration of buckled brace under cyclic loading is well documented. Thus the avoidance of brace buckling in EBFs permits stable hysteretic behavior under the most severe cyclic loading conditions. Note that the link not only limit brace forces, but also the load transmitted to the columns, permitting reliable design for column stability, and offering some possible advantages for difficult foundation design problems .

The ductility and energy dissipation capacity of EBFs is proved experimentally under cyclic lateral loads applied on the structures. This is observed, EBFs ability to sustain large deformations without strength loss which is an indicative of excellent energy dissipation capacity of eccentrically braced frame. This is due to buckling of brace is prevented and the link can sustain large deformations without strength loss.

The elastic lateral stiffness of an EBF will vary as a function of the link length e. When e=L, the frame has a moment resisting one and its elastic stiffness becomes minimal as shown in Figure 2.9. For e/L> 0.5 little stiffness is gained from the bracing. However, as the length of the link decrease, a rapid increase in stiffness occurs. Maximum stiffness develops when e=0, corresponding to a concentrically braced frame. When e=0, there is no link present to act as a fuse for brace member forces. In order to gain maximum possible frame stiffness, the links must be kept short but too short link has excessive inelastic deformations.

2.8.5. Concentrically Braced Frames

These are the type of bracings whose centroidal axis coincides with each other. They mainly increase the lateral stiffness of the frame which in turn increases the natural frequency and also decreases the lateral storey drift. However, increase in the stiffness may attract a larger inertia force due to earthquake. Further, the bracing increases the axial compression in the columns to which they are connected by decreasing the bending moments and shear forces in the column. And if, the bracings are omitted the bending moments and shear forces in columns increase but the axial compression in the columns to which they are connected is decreased. In the case of concrete building since reinforced concrete columns are strong in compression, it may not pose a problem to retrofit in RC frame using concentric steel bracings.

Concentrically braced frames have suitable lateral stiffness to prevent relative drift due to lateral load impacts resulting from earthquake. Such braces are part of relatively stiff systems and compatible with common needs of architecture with varied forms as shown (Figure 1.6). And also concentrically braced frames can be arranged in different forms such as cross, diametric v-shape, Chevron (inverted-v), K shape, etc. Those types of braces have not any link length between the connection points of bracing and beams that differs from eccentrically bracing type. It is a common phenomenon to use either steel or concrete materials for structural bracings as a lateral load resisting mechanisms in areas of high seismic zonal regions. Or it can be also used shear walls either at the periphery of the buildings or at the locations of lift as a core structure.

Generally, the use of steel concentrically bracing systems instead of Shear walls provides lower stiffness and resistance for a structure but it should not be forgotten that such a system has lower weight and more useful for architectural purposes.

For this research paper emphasis is given for concentrically and eccentrically type of steel bracing having eight different types of geometrical arrangements with similar cross section for comparison purpose. Different researchers‟ workout comparisons of the efficiencies of different forms of bracings, but their criteria of making assumptions for the selected group are not similar. Most Comparison of bracing was made by taking shape as the only criteria.

In any engineering problems formulation, criteria must be set for equal treatments of the phenomenon. Otherwise evaluations led to biased solutions and it creates also fallacy to have an optimum design type. If the designers/Engineers taking shape as the only criteria, the structure may be safe and stable but it may give unfair cost distribution to each type of bracings due to different weight of bracing members.

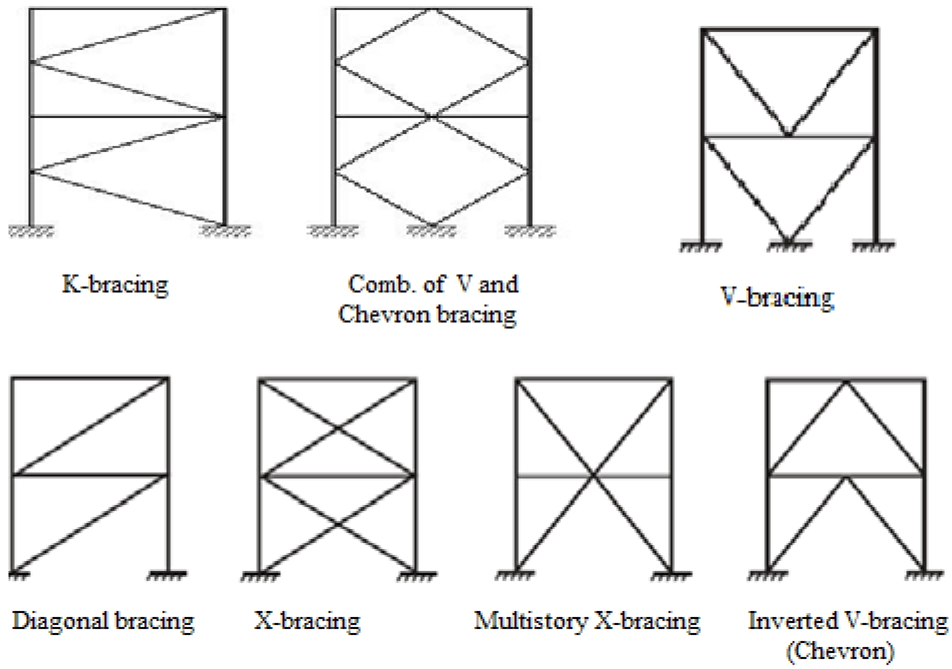


Figure 1.6 - Different types of concentrically braced frames except k-bracing and v- bracing which are eccentric bracings.

Braced frames and moment frames are the most widely used framing systems for steel construction in seismic regions. Compared to a moment frame, a braced frame offers high-lateral stiffness for drift control. In a CBF, the members (beams, columns and braces) with the centerlines meeting at a joint form a vertical truss system. Members in a CBF are subjected primarily to axial loads in the elastic range. The diagonal bracing members are designed to deform in elastically during a moderate or severe earthquake.

Braces in a conventional CBF are expected to buckle and yield during a significant seismic event. On the basis of a significant amount of research in the past few decades, seismic design provisions have been developed. In the AISC Seismic Provisions (2002), a conventional CBF can be designed as a Special CBF (SCBF) or as an Ordinary CBF (OCBF), depending on the ductility detailing requirements that are implemented into the system.

V or Inverted-V bracing is a popular configuration in the United States. Because one brace in a story is expected to buckle and lose a significant amount of compressive strength while the other brace is expected to yield during tension, the AISC Seismic Provisions require that for SCBFs the beam must be designed for an unbalanced vertical load at mid span. It has been suggested that the adverse effect of this unbalanced load be mitigated by using bracing configurations such as V and Inverted-V braces in alternate stories to create an X-configuration over two-story modules. The global design objective for energy dissipation in the case of Concentrically Braced Frames is to form dissipative zones in the diagonals under tension, and to avoid yielding or buckling of the beams or columns. Diagonals in compression are designed to buckle. The expected behavior for global mechanism in the case of a frame with chevron bracing (inverted V bracing) is shown in Figure 1.7.

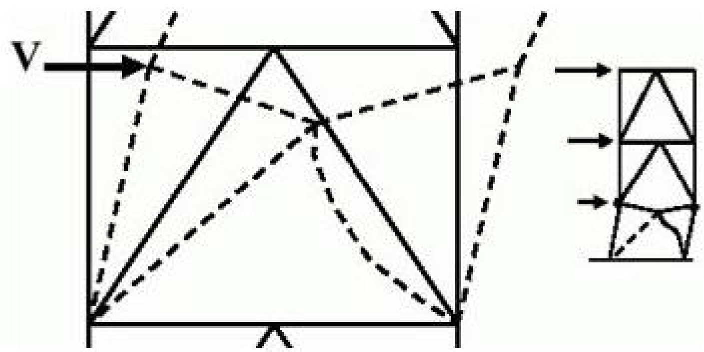


Figure 1.**7-** Chevron Brace Buckling

In this case, when the compression brace buckles, tension braces force doubles (before buckling has 50% of V in the tension brace and 50% of V in the compression brace). The vertical component of the tension brace axial force becomes a point load on the beam, pulling the beam down and possibly leading to hinging and buckling of the brace frame column.
When chevron bracing is used, the beam must be designed for an unbalanced load when the compression brace buckles. In this case the resulting brace frame beam design weighing more. By comparison, when a two story X brace is used, when the compression brace buckles at the first floor, the braces at the second floor prevents the brace frame beam from buckling and designing the beam for an unbalanced loading is not necessary .

The standard analysis of bracing frame is made assuming that: under gravity loading, only the beams and columns are present in the model and under seismic loading, only the diagonals in tension are present in the model.

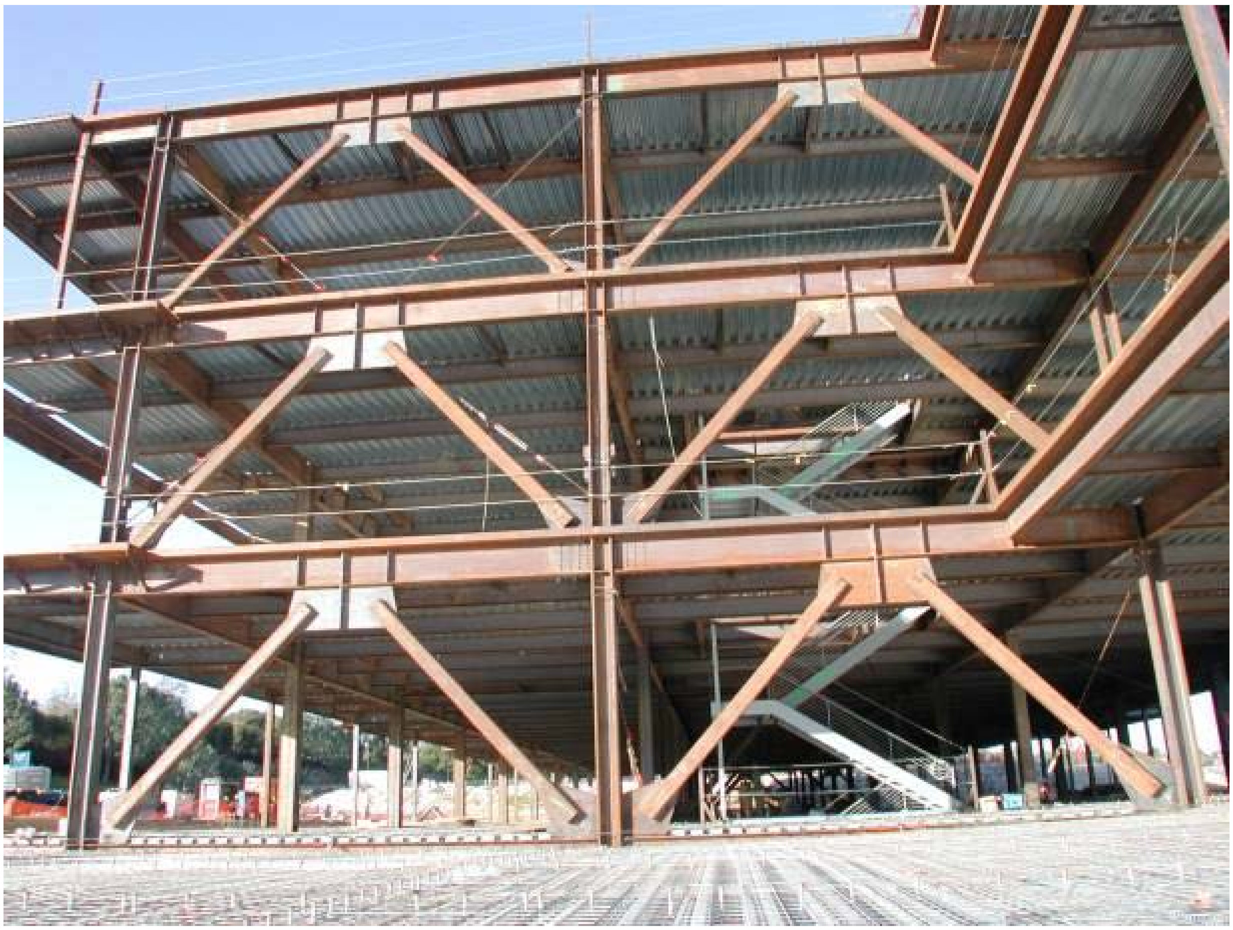


Figure 1.8- Concentric (chevron) Bracings (AISC 2004)

Figure 1.9. Concentric (combination of V and inverted V) Bracings (AISC 2004)

2.8.5.1. Performance of Concentrically Braced Frames

The design of a multi-story steel building under lateral loads is usually governed by system performance criteria (overall stiffness) rather than by component performance criteria (strength). An important task in the design of a tall steel building for structural designers is to select cost efficient lateral load resisting systems. Past studies reveals that pure rigid frame systems alone are not efficient in resisting lateral loads for tall steel buildings due to associated high costs. Thus, truss members such as diagonals are often used to brace steel frameworks to maintain lateral drifts within acceptable limits. In the absence of an efficient optimization technique, the selection of lateral bracing systems for multi-story steel frameworks is usually undertaken by the designer based on a trial and error process and previous experience. The optimal layout design of bracing systems is a challenging task for structural designers because it involves a large number of possibilities for the arrangement of bracing systems.

System performance is strongly influenced by aspects of brace behavior (Lehman et al. 2008). As Lehman proved in his experiment, Brace buckling places large inelastic demands on the brace at the middle of the brace, typically resulting in a plastic hinge at mid span as shown in Figure 2.0- (a). Brace buckling also places significant demands on gusset plate connections (Figure 2.0-(b) and adjacent framing members (Figure 2.0-(c). Limited cracking of the welds joining the gusset plate to the beams and columns generally is expected because of gusset plate deformation. These cracks normally initiate at story drifts in the range of 1.5 % to 2.0 %, but the cracks remain stable if the welds meet size and demand-critical weld requirements.

a) Brace buckling deformation b) Deformation of gusset plate c) local yielding in beam and column

Figure 2.0 - Various aspects of braced frame behavior

CBFs are also strong, stiff and ductile, making them ideal for seismic framing systems. The inelastic behavior of the brace provides most of the ductility, but in order to fully utilize the frame, the Connections and framing members must also be taken into account. Therefore, it is important to consider not only the performance of the brace when designing, but also the ability for the connections and the framing members to withstand the strength and deformation demands transferred from the brace during cyclic loading. Through these considerations, a maximum amount of energy can be dispersed before the system fails.

Cyclic testing of conventional braced frames shows that these braces buckle in compression and yield in tension. Plastic hinges occur after the brace has buckled and the stiffness and resistance of the frame decreases, illustrated in Figure 2.1. In Zone 0-A, the frame retains its elasticity, but the brace buckles at A, causing a plastic hinge to form in Zone A-B. Load reversal in Zones B-C, C-D and D-E cause the brace to become unstable, decreasing the effectiveness of the frame. This unstable behavior is evident in the unsymmetrical response seen in Figure 2.1 a. For this reason, Special Concentrically Braced Frames (SCBFs), with braces in opposing pairs, are used given the stable inelastic performance.

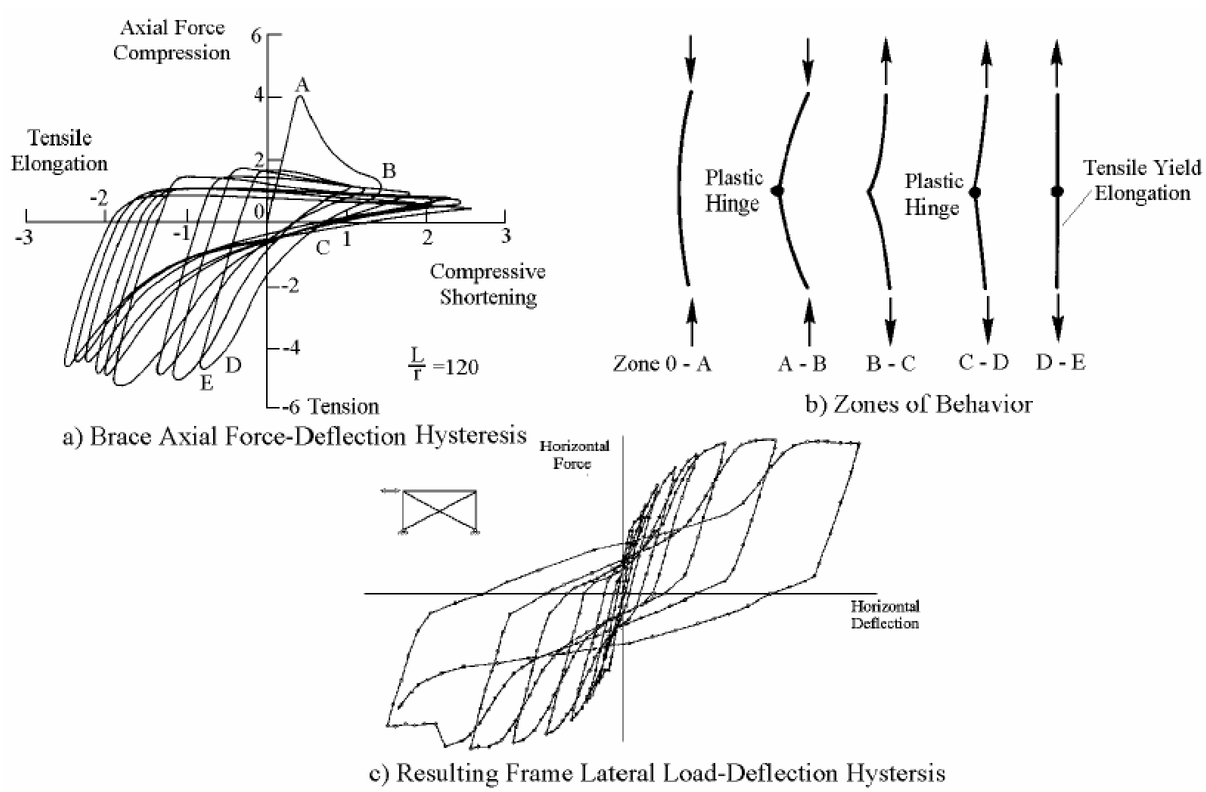
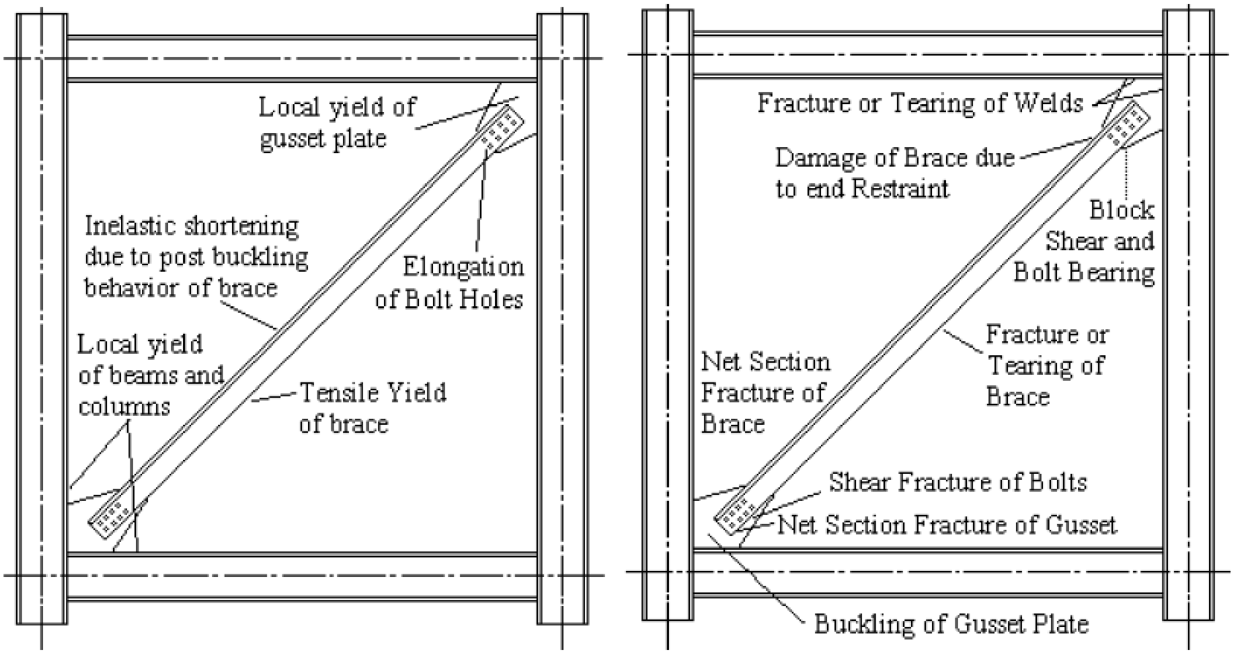


Figure 2.1 - Behavior of Special Concentrically Braced Frames

As per the code provision what the engineer is expected to do is that, the brace should fail before the connection does. The goal of the Performance-Based approach is to create a more detailed hierarchy of failures. A collection of permissible yield mechanisms and failure modes for a system can be identified. The permissible yield mechanisms are brace buckling and yielding, local yielding of the gusset plate, bolt-hole elongation, and the permissible failure modes include fracture or tearing of the brace. Unacceptable failure modes are buckling of the gusset plate or fracture of connection components such as bold or weld which is shown in the Figure below.



a) Yield mechanisms b) Failure modes

Figure 2.2 - Yield Mechanisms and Failure Modes for SCBF Components

Performance-Based Methods match the performance of a structure and the damage that is expected with varying levels of seismic activity. Figure 2.1 shows these possible relationships. The three performance levels are Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). As is expected structural damage increases with seismic levels and the permissible damage is more restricted with CP than IO.

2.8.5.2. Principles for Design of Steel Special Concentrically Braced Frames

The Special concentrically braced frame (SCBF) system is generally an economical system to use for low and medium rise buildings in areas of high seismicity. It is preferred over Special Moment Frames because of the material efficiency of CBFs and the smaller required beam and column depths. SCBFs are only possible for buildings that can accommodate the braces in their architecture. It also economically develop the lateral strength and stiffness needed to assure serviceability and structural performance during the smaller as well as frequent earthquakes, but the inelastic deformation needed to ensure life safety through collapse prevention during extreme earthquakes is dominated by tensile yielding of the brace, brace buckling, and post buckling deformation of the brace.

The ductility and inelastic deformations required by this second design goal vary in magnitude depending upon the seismic hazard level and the seismic design procedure. For areas of low seismicity, ASCE 7 allows steel framing systems to be designed with a Response Modification Factor, *R*, of 3.0 with no special detailing requirements to improve ductility. ASCE 7 also allows the use of Ordinary Concentrically Braced Frames (OCBFs). However, SCBFs are designed with relatively large *R* factors, and as a consequence are expected to experience relatively large inelastic deformation demands during extreme ground shaking. A story drift of approximately 2.5 % is commonly assumed as a target inelastic deformation to be achieved by SCBFs prior to brace fracture. As a result, ductile detailing and proportioning requirements are needed to ensure that SCBFs can achieve the required inelastic deformations. Corresponding inelastic flexural deformation in beams, columns, and connections will occur during these large inelastic excursions. The inelastic deformations in the beams and columns are not primary effects because they are not specific goals of the design process. Nevertheless, they influence the seismic performance of SCBFs and contribute to the cost of repair. Local slenderness limits for beams and columns are required by AISC 341 in recognition of these local inelastic deformations.

The configuration of braces affects system performance. Multiple configurations of bracing can be used, and these configurations are identified in Figure 1.6 and 1.5. Braces buckle in compression and yield in tension. The initial compressive buckling capacity is smaller than the tensile yield force, and for subsequent buckling cycles, the buckling capacity is further reduced by the prior inelastic excursion. Therefore, bracing systems must be balanced so that the lateral resistance in tension and compression is similar in both directions. This means that diagonal bracing or chevron bracing must be used in matched tensile and compressive pairs. As a result, these bracing must be used in opposing pairs to achieve this required balance. Other bracing configurations, such as the X-brace, multistory X-brace and chevron brace directly achieve this balance. X-bracing is most commonly used with light bracing on shorter structures. Research shows that the buckling capacity of X-bracing is best estimated by using one half the brace length when the braces intersect and connect at mid section. However, the inelastic deformation capacity of the X-braced system is somewhat reduced from that achievable with many other braced frame systems because the inelastic deformation is concentrated in one-half the brace length because the other half of the brace cannot fully develop its capacity as the more damaged half deteriorates. The compressive buckling resistance of most other brace configurations is best estimated by considering true end-to-end length of the brace with an effective length factor, K, of 1.0 (i.e., neglecting rotation stiffness of the brace-to-gusset connection.) Inelastic deformation of the brace dominates the inelastic performance of SCBFs during moderate and large earthquakes, and fracture of the brace at mid-length is clearly the anticipated initial failure mode of the braced frame system. A number of brace design issues affect the inelastic deformation and ultimate fracture of the brace.

2.8.6. Design Approach for Bracing Systems

Braced-frame members are designed to resist the forces specified by the building code based on the type of structural system selected and the location of the building site relative to various faults and seismic source zones, as determined from seismic risk or zonation maps. Under the requirements of the AISC Seismic Provisions the brace members of an ordinary braced frame, except chevron configurations, are designed for the force corresponding to the application of the specified base shear force per the applicable building code. In the case of chevron or V braces, the design force is increased by 150% .This is due to when compression brace buckles only tension brace will carry full loads. However, this requirement of 150% increase in the design force is not applicable if the chevron is designed as a special concentric brace frame (SCBF).

All bracing connections are required to be capable of resisting the maximum expected force that could be delivered to them by the bracing configuration. The design intent is that the strength ofall of the brace frame components (beams, columns, connections) be larger than the expected maximum capacity of the brace member. By ensuring this, the failure of a braced-frame system is intended to be controlled by yielding and buckling of the braces only, not the other elements ofthe frame. As soon as braces yield in tension or buckle in compression, they start to classify under increasing lateral loads. As full classification occurs, the stiffness and load-carrying capacity of the brace is limited and, therefore, the load that may be attracted to the brace frame as a whole is limited. As a result, only the brace member will be damaged and will require repairs after an earthquake, whereas all other components of the braced frame will be undamaged and require no repair. Note that as previously discussed; the design requirements for ordinary V- and Chevron-braced frames are not adequate to accomplish this objective, as the beams at the apex of the V or chevrons are vulnerable to damage.

The AISC Seismic Provisions require that brace connections are designed for the lesser of the following forces:

- The strength of the brace in axial tension.
- An over strength factor (**Ω_o)** times the design force in the brace including gravity loads.
- The maximum force that can be transferred to the brace by the system, considering other limiting factors, such as the capacity of diaphragms to transfer shear forces to the braced frames. For the section to be safe, the yield resistance Npl, Rd of the bracing diagonals should be greater than the axial tension force NEd computed under the seismic action effect: Npl, Rd > Ned For each bracing diagonal, the ratio of the yield resistance provided Npl, Rd to the resistance required NEd is determined: Ω i = Npl, Rd, i / NEd,.

These ratios ̀Ω i represent the excess capacity of the sections with respect to the minimum requirement and are therefore called „section over-strength‟. In order to achieve a global plastic mechanism the values of ̀Ω i should not vary too much over the full height of the structure, and a homogenization criterion is defined; the maximum ̀Ω i should not differ from the minimum by more than 25%. As the diagonals are effectively ductile „fuses‟, the beam and column design forces are a Combination of:

- ✓ The axial force NEd, G due to gravity loading in the seismic design situation.
- ✓ The axial force NEd, E due to seismic action amplified by the „over-strength‟ of the Diagonal, which is found by multiplying the section „over strength‟ factor" by the material „over strength‟ γov (when applying so called capacity design).

The axial load design resistance Npl, Rd of the beam or the column, which takes into account interaction with the design bending moment MED in the seismic design situation, should satisfy:

Npl, Rd (MED) > NED, G +1.1γov ̀Ω NEDE

2.8.7. Preference of Bracing Location

Bracing can be located at different location of the structure. It can be located at the center or any sides of the building but its resisting capacity and efficiencies for the stability and torsion capacity of the structure is completely different. As the free encyclopedia for UK steel construction information said it is preferable to locate bracing at or near the extremities of the structure [8], in order to resist any torsion effects.

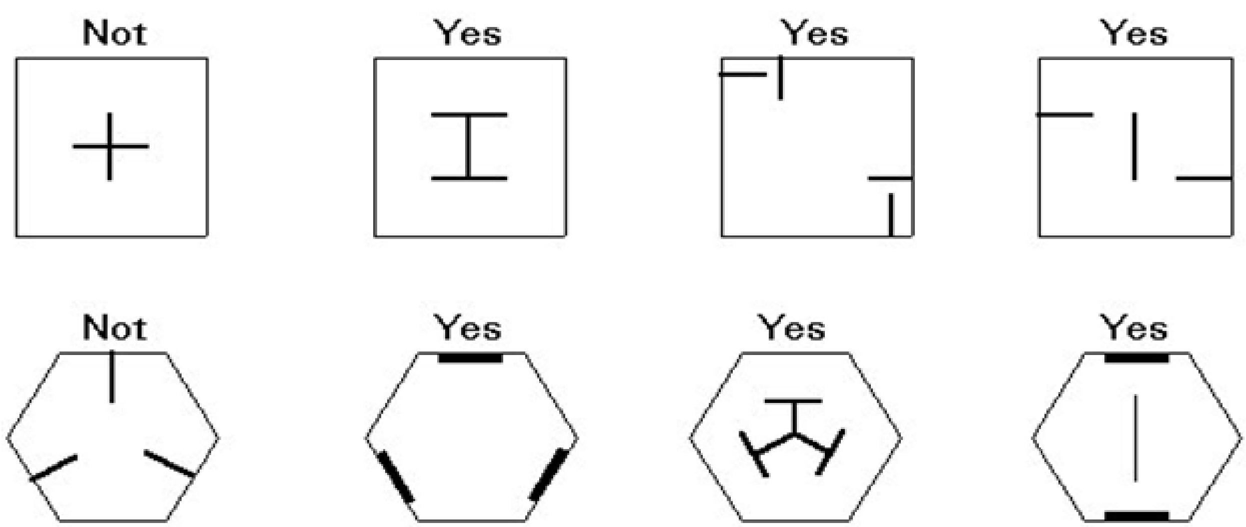


Figure 2.3 - Bracing System Location (AISC 1999)

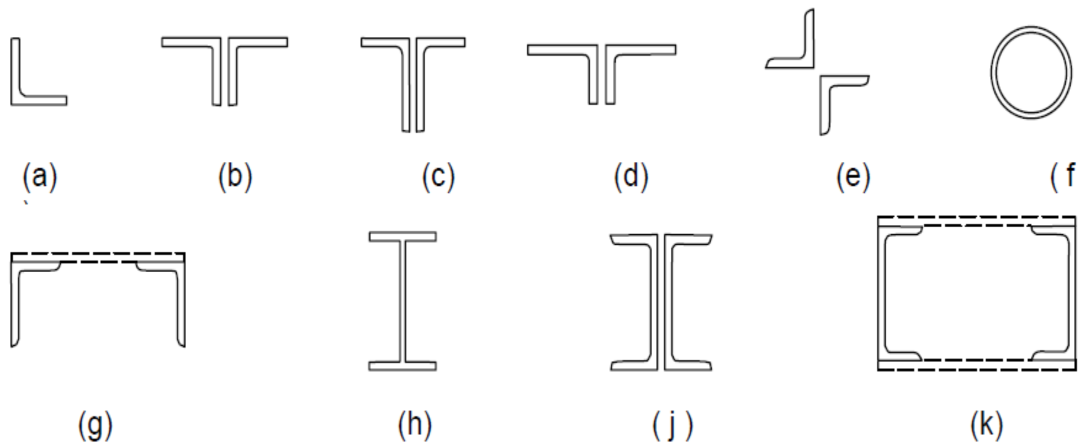


Figure 2.4 - Bracing section (AISC 1999)

Braced frames are most effective at the building perimeter, where they can control the building's torsion response. ASCE 7 allows buildings to be considered sufficiently redundant (and thus avoid a penalty factor) with two braced bays on each of the presumed four outer lines (assuming a rectangular layout). Such a layout is good for torsion control as well. In the same way; in mid-rise or high-rise buildings, braced frames are often used in the core of the structure, with a perimeter moment frame used to provide additional torsion resistance. Where possible, bracing members inclined at approximately 45° are recommended. This provides an efficient system with relatively modest member forces compared to other arrangements, and means that the connection details where the bracing meets the beam/column junctions are compact. Narrow bracing systems with steeply inclined internal members will increase the sway sensitivity of the structure. Wide bracing systems will result in more stable structures. But the wider bracing affects the aesthetical values of the building and it may prevent door and windows openings. This obstruction can be minimized by providing V-bracing for windows opening and chevron bracing (inverted V-bracing) for door opening. The table below gives an indication of how maximum deflection varies with bracing layout, for a constant size of bracing cross section.

Table 1.1 Comparisons of Bracing efficiencies at different angle of bracing inclination.

Bracing efficiency			
Story height	Bracing width	Angle from horizontal	Ratio of maximum deflection (compared to bracing at 34°)
h	2h	26°	0.9
h	1.5h	34°	1.0
h	h	45°	1.5
h	0.75h	53°	2.2
h	0.5h	63°	4.5

CHAPTER 3. MODELING AND LOADING OF STRUCTURAL SYSTEM

3.1 Modeling Software, ETABS

ETABS (Extended Three-Dimensional Analysis of Building Systems) is special purpose analysis and design program developed specially for buildings. Original development of TABS 30 years back led to the development of the today's ETABS. Early releases of ETABS provided input, output and numerical solution that took into consideration the characteristics unique to building type structures, providing a tool that offered significant savings in time and increased accuracy over general purpose programs.

As computers and computer interfaces evolved, ETABS added computationally complex analytical options such as dynamic nonlinear behavior, and powerful CAD-like drawing tools in a graphical and object-based interface.

ETABS offers the widest assortment of analysis and design tools available for the structural engineer working on building structures. The following list represents just a portion of the types of systems and analysis that ETABS can handle easily:

- Multi-story commercial, government and health care facilities
- Parking garages with circular and linear ramps
- Staggered truss buildings
- Buildings with steel, concrete, composite or joist floor framing
- Buildings based on multiple rectangular and/or cylindrical grid systems
- Flat and waffle slab concrete buildings
- Buildings subjected to any number of vertical and lateral load cases and combinations, including automated wind and seismic loads
- Multiple spectrum load cases, with built-in input curves
- Automated transfer of vertical loads on floors to beams and walls
- P-Delta analysis with static or dynamic analysis
- Explicit panel-zone deformations
- Construction sequence loading analysis

- Multiple linear and nonlinear time history load cases in any direction Foundation/support settlement
- Large displacement analysis
- Non linear static pushover
- Buildings with base isolators and damper
- Floor modeling with rigid or semi-rigid diaphragms
- Automated vertical live load reductions

3.1.1 Physical Modeling Terminologies in ETABS

In ETABS objects, members, and elements are often referred. Objects represent the physical structural members in the model. Elements, on the other hand, refer to the finite elements used internally by the program to generate the stiffness matrices. In many cases objects and physical members will have a one-to-one correspondence, and it is these objects that the user draws in the ETABS interface.

In ETABS, objects or physical members drawn by users, are typically subdivided into the greater number of finite elements needed for the analysis model, without user input.

3.1.2 Structural Objects

ETABS uses objects to represent physical structural members. The following objects are available in ETABS:

- ➢ Point objects
- ➢ Line objects and
- ➢ Area objects

3.2 Problem modeling

For non linear analyses, a three dimensional T shape steel building has been selected. Because irregular buildings have tortional problems. Thus bracings which are efficient for such structure can be used for other structure with different geometry without challenge. The building considered for analysis is twenty-five T plan steel building of asymmetrical in plan where as symmetrical in elevation on 60mx60m plan size. Columns are spaced at 4m interval in both directions and its corresponding story height is 4m. It is also assumed that to support the deck slab secondary steel beam is provided with a spacing of one meter at each floor level. The bracing is assumed to be a channel section which is provided at the periphery of the building at each floor level.

The lateral seismic loads to be applied on the building are based on Ethiopian Building Code of Standards. The study is performed for seismic zone 4 in Adama city as per the code provision and with basic wind speed of 22 m/s as per the code provision. The frames are assumed to be firmly fixed and the soil structure interaction is neglected. The load combinations and other design parameters associated with the steel structure are as per EBCS 8.

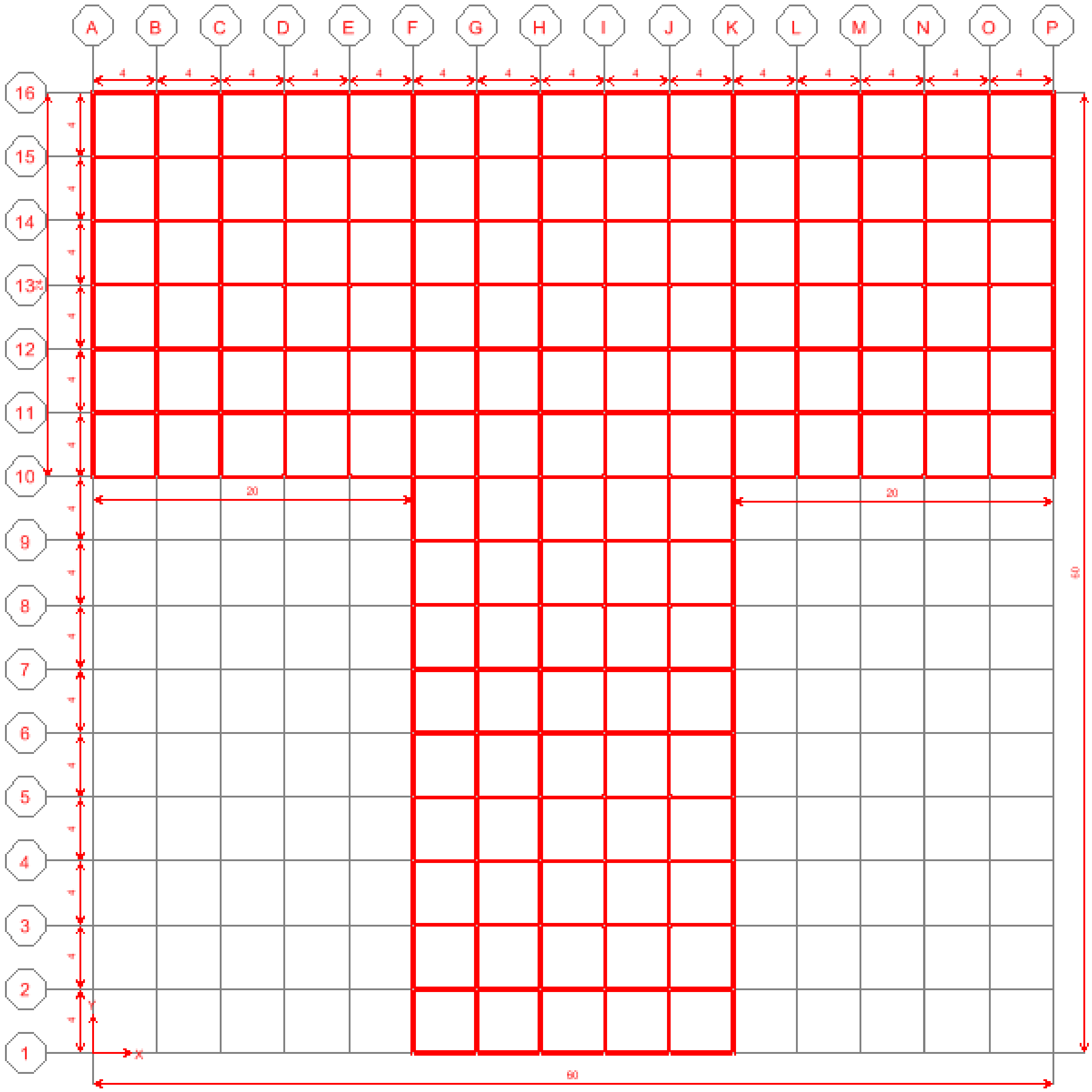


Figure 2.5- Layout plan of steel building

In order to evaluate different bracing systems, prior to going into any action for assessment, model with different bracing systems must be considered. The model has sixteen frames consisting of columns and beams running along the building longitudinally. The transverse beams connect the sixteen longitudinal frames. In case of braced frame, all Sixteen external frames are braced by bracing systems placed only at corner panels and at middle Panels along longitudinal direction. In this study two different types of structural bracing systems have been considered. They are the eccentric bracing systems and Concentric bracing systems.

Beams and columns are of universal I steel sections and each bracing system has been analyzed using channel section. For the simplification of the study the same sections has been used for all bracing systems. Depending on the number of storey of the building, for all steel members nominal steel grade of Fe510 has been used. The following table 2 represents a brief summary of key structural features of the building. The building has been modeled using ETABS 9.7.1 software package and then a linear static analysis has been performed on the same structure for both concentric and eccentric type of bracings.

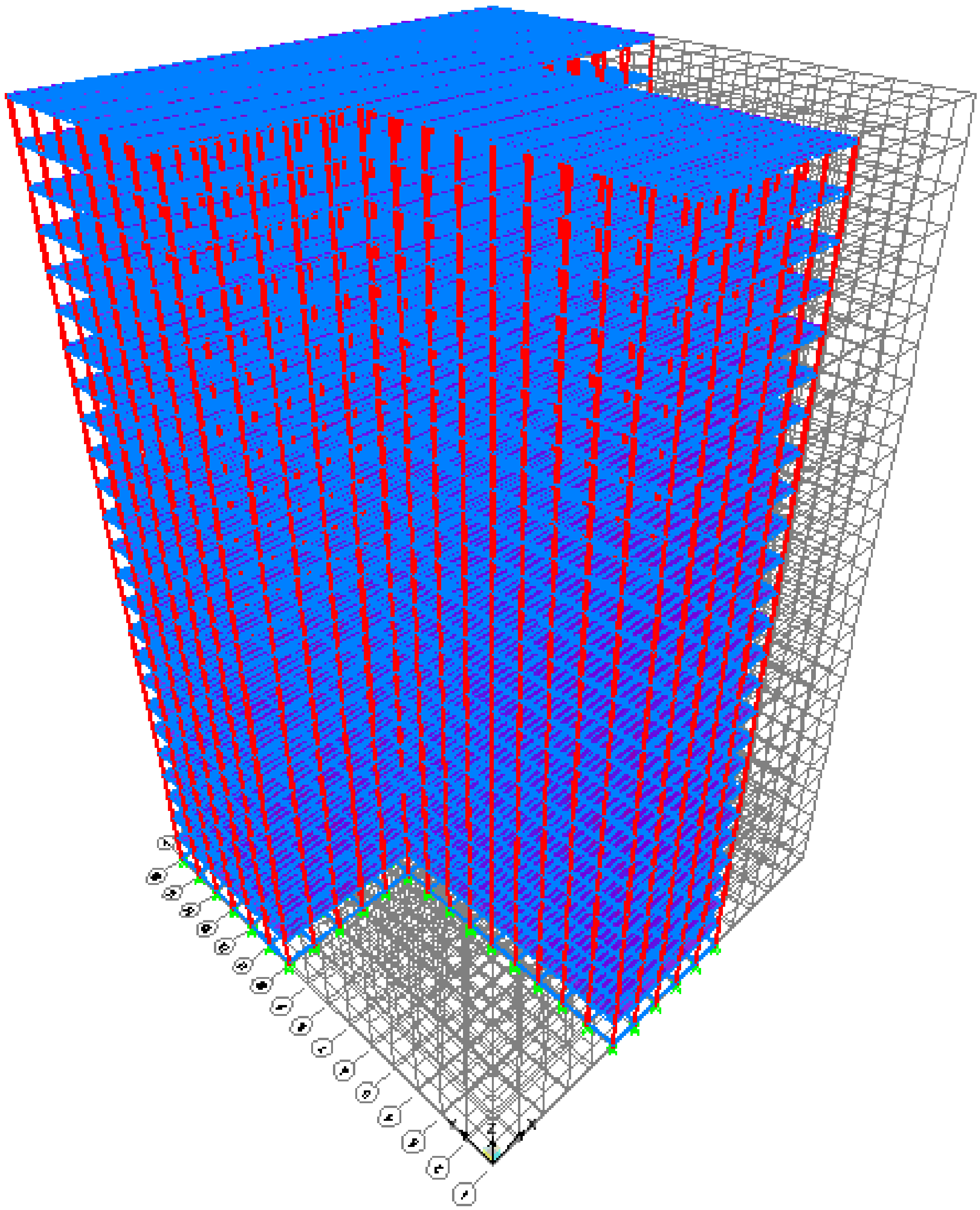


Figure 2.6- 3D view of Steel building without bracing

In this study, different forms of concentrically and eccentrically braced frames are taken to evaluate the performance of bracings by setting equal weight /volume as the criteria which has to be applied for each system of bracings. Each bracing is tested on plan irregular twenty-five storey steel building. One model type is modeled for each bracing types to investigate the behavior for each bracing systems.

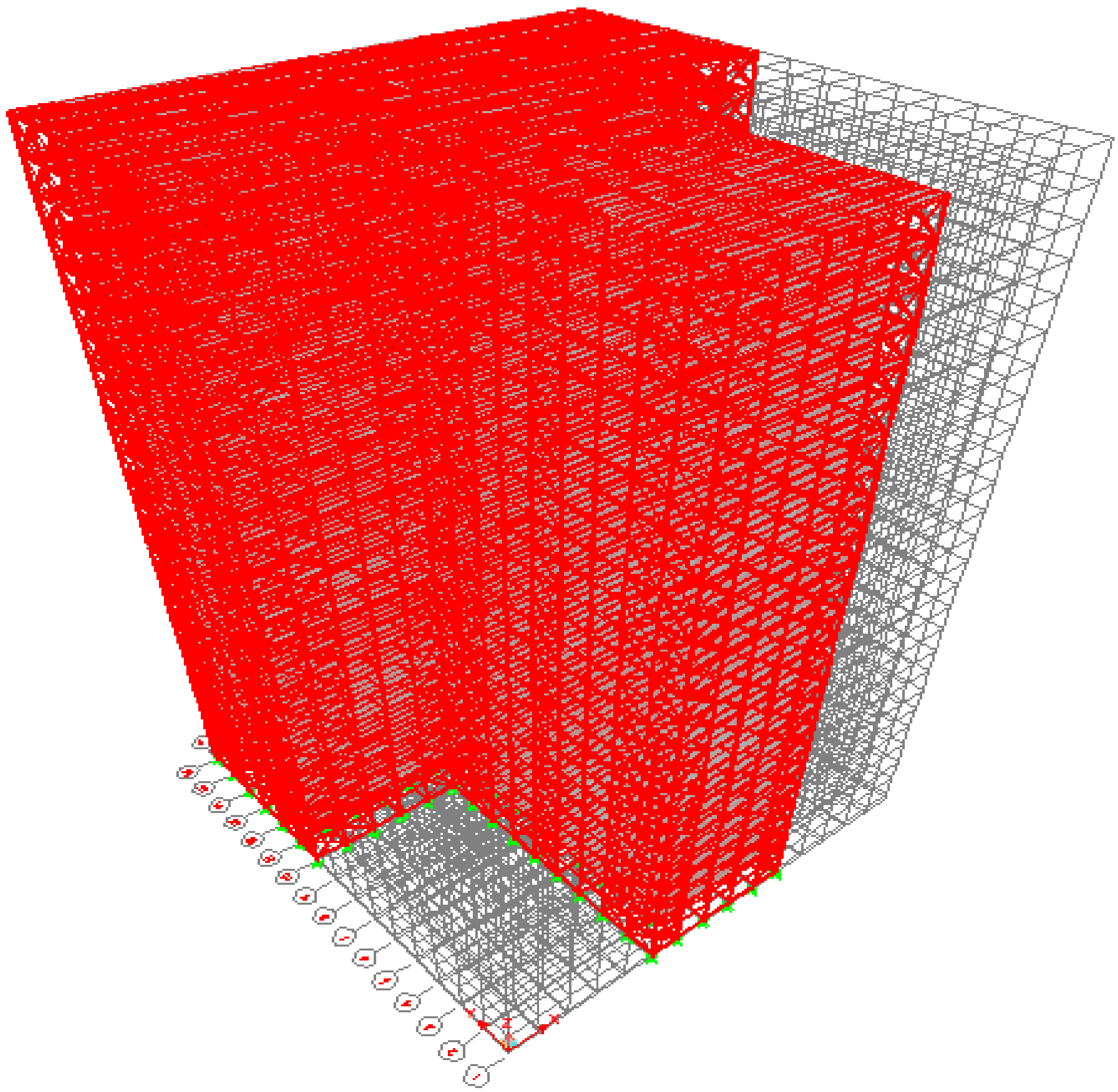


Fig 2.7 - 3D view of Steel building with bracing

The details of frames geometry and location of bracings are arranged as the following**:**

Different concentric types of braces and eccentric braces have been implemented in one frame with twenty-five storey frame and analyzed with respect to un-braced reference model. The use of three bays is a good choice for the efficient placement of the selected bracing type which consists of X bracing V-bracing, combination of V and inverted V-bracing, knee bracing, diagonal bracing, three eccentric types of bracing systems except V bracing which is eccentric type are provided within the frame central bays of the exterior parts of the building as shown in Figures below.This aids to differentiate the behavior of bracings in high rise irregular steel building.

Thus four major type of bracing systems of concentrically braced and four eccentrically braced frame types are modeled, analyzed and compared with respect to un-braced reference model.

3.2.1 The type of Concentric and Eccentric bracing systems used for 25 storey building.

i. X bracing system (Model 1)
ii. Combination of V and Inverted V bracing system (Model 2)
iii. Diagonal bracing system (Model 3)
iv. Knee bracing system (Model 4)
v. Eccentric bracing systems (Model 5, Model 6, Model 7, Model 8).

3.2.2 Key Structural Feature

Table 1.2 Geometric data for modeling and analysis of assumed buildings

Type f structure	Steel moment resisting frame
Number of stories	G+25
Height of each story	4.00m
Space of columns	4.00m
Bottom story height	0.80m
Type of building	Industrial or commercial
Seismic zone and location	4 (Adama city)
Basic wind speed	22m/s

An I section of serial size (356x171) mm @ 51Kg/m is used throughout the structure as a beam member. To withstand the load coming from beams, wall loads and slab load, a column of serial size (203x203) @ 71 Kg/m is used initially for analysis. Channels sections are also used as bracing elements. The channel section used for present study is (178x76) mm serial size @ 20.84Kg/m.

The type of steel grade with its corresponding nominal yield strength f_y and ultimate tensile strength f_u that used for analysis purpose is based on the number of stories as shown in table 1.4.

Table 1.3. The cross section and weight of the structural member

Member	Type of section	Serial size(mm)	Weight(kg/m)
Beam	Universal beam(I)	356x171	51
Column	Universal column(I)	203X203	71
Bracing	channel	178x76	20.84

Table 1.4 Steel Grade taken from EBCS 3 for the given storey

No of Storey	Nominal steel grade	Fy(Mpa)	Fu (Mpa)
25	Fe 510	355	510

Note: f_y and f_u values are for thickness of section t < 40mm.

3.2.3 Loading consideration in ETABS software

For analysis of this steel building, different loads are considered. These are self weight of the structure, external wall load at the periphery of the building which is called cladding load, superimposed load from fixed furniture, live load, and earthquake loads are considered. These loads are taken by assuming the building is to produce similar service at each level of the floors systems. Because the main target of the study is to evaluate and identify the most effective types of bracing, from the given concentrically types of bracings and eccentrically types of bracings.

The building is subjected to the following loads as per EBCS 1 (Basis of design and Action on the structure).

Floor finish = 1.0 KN/m^2

Live and Dead loads = 4 KN/m^2

Live load on roof =2.0 KN/m^2

Table 1.5 Live and Dead Load data acting on the building according to EBCS1

Density of brick wall	20KN/m3
Dead load on slab	4KN/m2
Live load on slab	4KN/m2
Thickness of wall	0.2m
Wall load load on beams(external wall load)	12KN/m
Earthquake load	As per the code provision

The imposed load from external walls 12 KN/m acts at each story of the external frames of the building considered due to similar purpose is assumed for all stories.

3.2.3.1 Vertical loads

a) Roof Floor

- Roof cover,EGA-500 (0.6mm thick)

  Unit weight = 4.71Kg/m^2 = 47.1N/m^2 = 0.0471KN/m^2

  To be increased by 20% for laps and fastenings

  0.0471 X x1.2 = 0.06KN/m^2
- Ceiling and support condition

  Unit weight = 0.7Kg/m^2/mm

  Using 8mm thick ceiling material

  0.7 X 8 = 5.6Kg/m2 = 0.056KN/m^2

  To be increased by 300% for support conditions

  0.056 X 3 = 0.17KN/m^2

  Total dead load = 0.06 +0.17 = 0.23 KN/m^2

  Total live load = 1.0 KN/m^2 for maintenance and construction conditions.
- Load on purloin (KN/m)

  Purloin spacing = 1.1m

Dead load on purloin = 0.23 KN/m^2 X 1.1m = 0.25 KN/m

Total live load on purloin = 1.1 KN/m^2 X 1.1m = 1.1 KN/m

Finally total dead load on roof = **0.23 KN/m^2** and dead load on purloin **0.25 KN/m.**

Live load on roof = **1.0 KN/m^2** and live load on purloin = **1.1 KN/m**

3.2.3.2 Seismic load

Base shear force

$F_b = S_d(T_1)m\lambda$….EBCS EN 1998-1-1:2013 section 4.3.3.2.2

$S_d(T_1)$ is the ordinate of the design spectrum at period T_1(to be filled on the analysis software.

$T_1 = C_t \times H^{3/4}$

$C_t = 0.085$

H =100m

$T_1 = 0.085 \times 100^{3/4}$m = 2.688

For soil type B (EBCS EN1998-1-1:2013 section 3.1.2), the values of S, T_B,T_C and T_D are given in EBCS EN1998-1-1:3.2.2 using type one elastic response spectra.

S = 1.2, T_B = 0.15, TC = 0.15, TD= 0.15

Since T_D (1.2) < T = 2.688

$$S_d(T_1) = \{ a_g S \frac{2.5}{q}\left(\frac{T_C T_D}{T^2}\right)$$

$$\geq \beta a_g$$

Where a_g = 0.2(design ground acceleration)

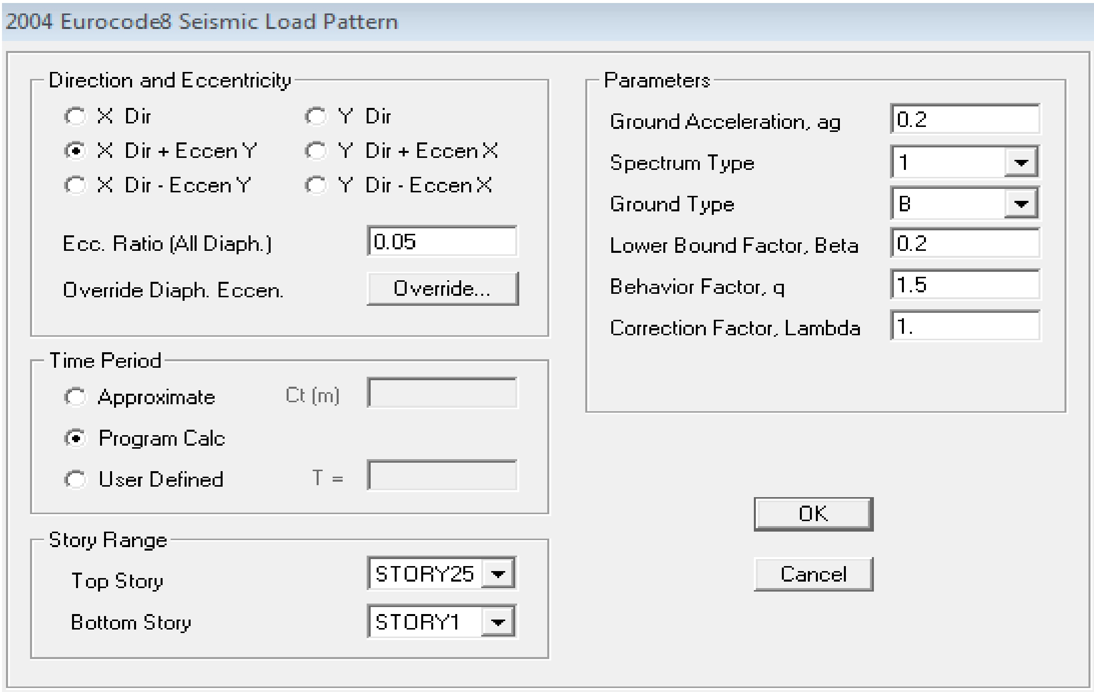
β = 0.2 (lower bound factor for the horizontal design spectrum).

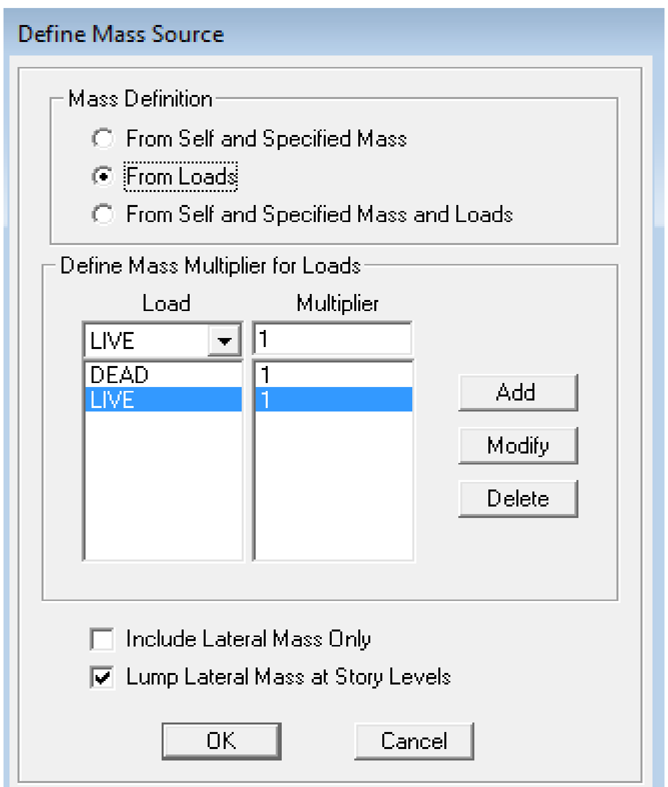
q = 1.5 (behavior factor)

$$S_d(T_1) = \{ 0.2 \times 1.2 \times \frac{2.5}{1.5}\left(\frac{0.5 \times 0.15}{2.688^2}\right)$$

$$\geq 0.2 \times 0.2$$

$$\mathbf{S_d(T_1) = 0.00415}$$


2004 Eurocode8 Seismic Load Pattern
Direction and Eccentricity
X Dir
Y Dir
X Dir + Eccen Y
Y Dir + Eccen X
X Dir - Eccen Y
Y Dir - Eccen X
Ecc. Ratio (All Diaph.)
0.05
Override Diaph. Eccen.
Override...
Time Period
Approximate
Ct (m)
Program Calc
User Defined
T =
Story Range
Top Story
STORY25
Bottom Story
STORY1
Parameters
Ground Acceleration, ag
0.2
Spectrum Type
1
Ground Type
B
Lower Bound Factor, Beta
0.2
Behavior Factor, q
1.5
Correction Factor, Lambda
1.
OK
Cancel


Define Mass Source
Mass Definition
From Self and Specified Mass
From Loads
From Self and Specified Mass and Loads
Define Mass Multiplier for Loads
Load
Multiplier
LIVE
1
DEAD
1
LIVE
1
Add
Modify
Delete
Include Lateral Mass Only
Lump Lateral Mass at Story Levels
OK
Cancel

CHAPHTER 4. ANALYSIS AND COMPARISON OF ANALYSIS RESULTS OF BRACING SYSTEM FOR SEISMIC LOADS

4.1. Introduction

The lateral load analysis of this study is based on EBCS codes design manuals. As per EBCS code 8, the horizontal design forces are defined from maximum acceleration of the structure, under the expected earthquake, that is represented with the acceleration spectrum of the structure. The starting point is an elastic response spectrum, which is reduced with factors that take into consideration the ability of structure to absorb seismic energy through rigid deformation. In the horizontal plane, the seismic action acts simultaneously and independently in two orthogonal directions that have the same response.

Ethiopian Building Code of standards suggests two different design spectrums:

Type 1 for the more seismically active zones (zone 4), and

Type 2 for less seismically active zones (zone 1and 2).

In this study, Type1 design spectrum is selected in order to notice the effect of earthquake on each bracing systems which may give maximum lateral displacement. In addition, there are also different parameters that are considered as an input for analysis. Of which behavior factor (q) is one factor that affect the analysis result. Depending on the dissipation behavior of the structure, behavior factor values varies from bracing to bracings. K-bracing types are less dissipative behavior compared to other concentrically bracing (V and Chevron types of bracings). Behavior factor values which satisfy regularity in elevation are shown in Table 1.6 blow.

Table 1.6 Upper limits of reference values of behavior factors for systems regular in elevation

STRUCTURAL TYPE	Ductility class	
	DCM	DCH
a) Moment resisting frames	4	$5\alpha u/\alpha 1$
b) Frames with concentric bracings Diagonals bracings	4u	4
c) Frames with eccentric bracings	4	$5\alpha u/\alpha 1$
d) Inverted pendulum	2	$2\alpha u/\alpha 1$
e) Structures with concrete cores or concrete walls	See section 5	
f) Moment resisting frame with concentric bracing	4	$4\alpha u/\alpha 1$
g) Moment resisting frames with in fills Unconnected concrete or masonry in fills, in contact with the frame	2	2
Connected reinforced concrete in fills	See section 7	
In fills isolated from moment frame (see moment frames)	4	$5\alpha u/\alpha 1$

As per EBCS 8 ground types are classified under five types by considering deep geological event and the influence of local ground conditions on the seismic action shown in Table 1.7.

Table 1.7 Ground type classifications as per EBCS code 8.

Ground Type	Description of stratigraphic profile	Parameters		
		Vs,30(m/s)	NSPT below 30cm	30cm cu (Kpa)
A	Rock or other rock-like geological formation, including at most 5m of weaker material at the surface	>800	-	-
B	Deposit of very dense sand, gravel or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250
C	Deep deposit of dense or medium- dense sand, gravel of stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70=250
D	Deposit of loose-to-medium cohesionlesss soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	180<	15<	<70
E	A soil profile consisting of a surface alluvium layer with vs values of type C or D and thickness varying between about 5m and 20m, underlain by stiffer material with vs> 800m/s			

For this research the following parameters are considered for earthquake analysis as per EBCS code 8-design of structures for earthquake resistance using ETABS version 9.7.1.

Table 1.8 General Parameters considered during Analysis

S.No	Parameters	Values
1	Design Ground Acceleration, ag	0.2g (as per EBCS 8)
2	Design spectrum type	1 (as per EBCS 8)
3	Ground type	B (as per EBCS 8)
4	Behavior factor as per the code of EBCS 8.	2 for K-bracing
		4 for other bracing
5	Accidental eccentricity	0.05

4.2 Earth quake analysis

The lateral loads are applied at ± $0.05l_b$ to include accidental tortional eccentricities resulting in the following four load cases:

- EQXRT (corresponds to center of mass at (Xg-rt/Yg)
- EQXLT(corresponds to center of mass at (Xg-lt/Yg)
- EQYRT(corresponds to center of mass at (Yg-rt/Xg)
- EQYLT(corresponds to center of mass at (Yg-lt/Xg)

4.2.1 Design Load combinations

The values of actions which occur simultaneously are combined as follows:

i) Persistent and transient situation

COMB 1 = 1.3G_k +1.6 Q_k

ii) Seismic situation

General format: $\sum G_{kj} + A_{Ed} + \sum \varphi_{2i} \times Q_{ki,}$ which can be approximated using COMB 1as:

0.75X COMB 1± A_{Edx} or 0.75 XCOMB 1± A_{Edy}

4.2.2 Equivalent static analysis

There are total of eight load combinations for seismic situation (COMB 2 to COMB9) corresponding to the four load combinations above whose seismic actions A_{Ed} have each two alternative lines of actions to take into account the effects of accidental tortional eccentricity.

4.2.3 Dynamic response spectrum analysis

COMBRSX – 0.75 X COMB1 +Specx +0.3xSpecy

COMBRSY – 0.75 X COMB1 +Specy +0.3xSpecx

Finally envelopes have been evaluated for the purpose of determining the design action effects at the critical regions.

ENVEX –Envelope of load combinations with seismic actions in the x- direction.

ENVEY - Envelope of load combinations with seismic actions in the y- direction.

ENVELOPE – Envelope of COMB1, ENVEX, and ENVEY

After analyzing and designing the building for seismic load as per Ethiopian Building Code of Standard, the universal column section for all exterior columns are designed as (400X200X8X13) mm and all interior columns are designed as (350X175X11) mm for the entire storey.

The following models of four different concentrically bracings and four different eccentrically bracings are subjected for seismic excitation in both orthogonal directions on the 25- story T shape irregular steel building, and analyzed using the finite element analysis software of ETABS non linear version 9.7.1 with an extended 3D analysis of building systems.

From the ETABS analysis result, due to lateral earthquake load effect, the frame produces different lateral displacement for each type of bracing model with bracings provided on the corner and middle bays of the elevation view. The following models (1, 2, 3 and 4) are of concentric bracings.

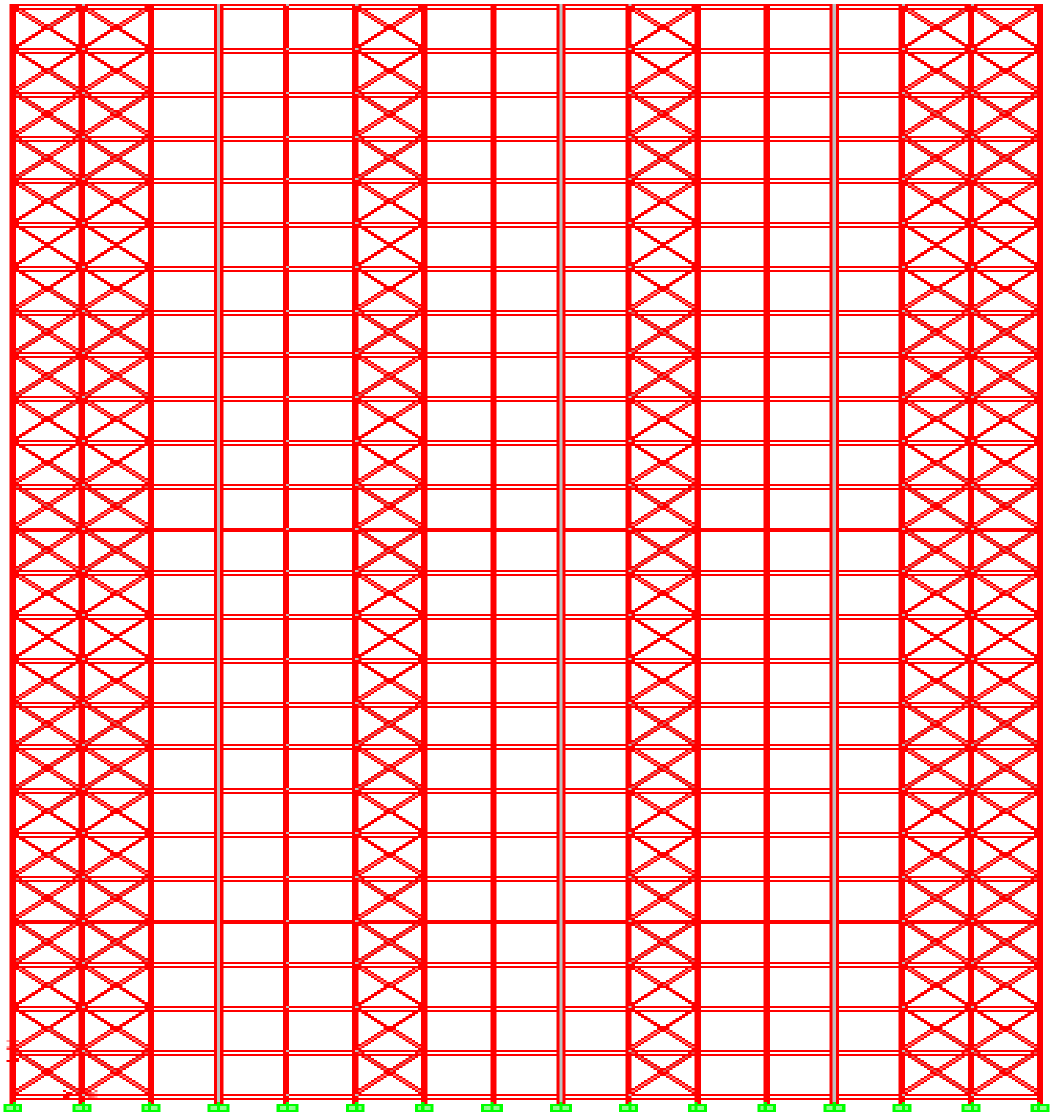


Figure 2.8- Concentric X bracing system along axis 16. (Model 1)

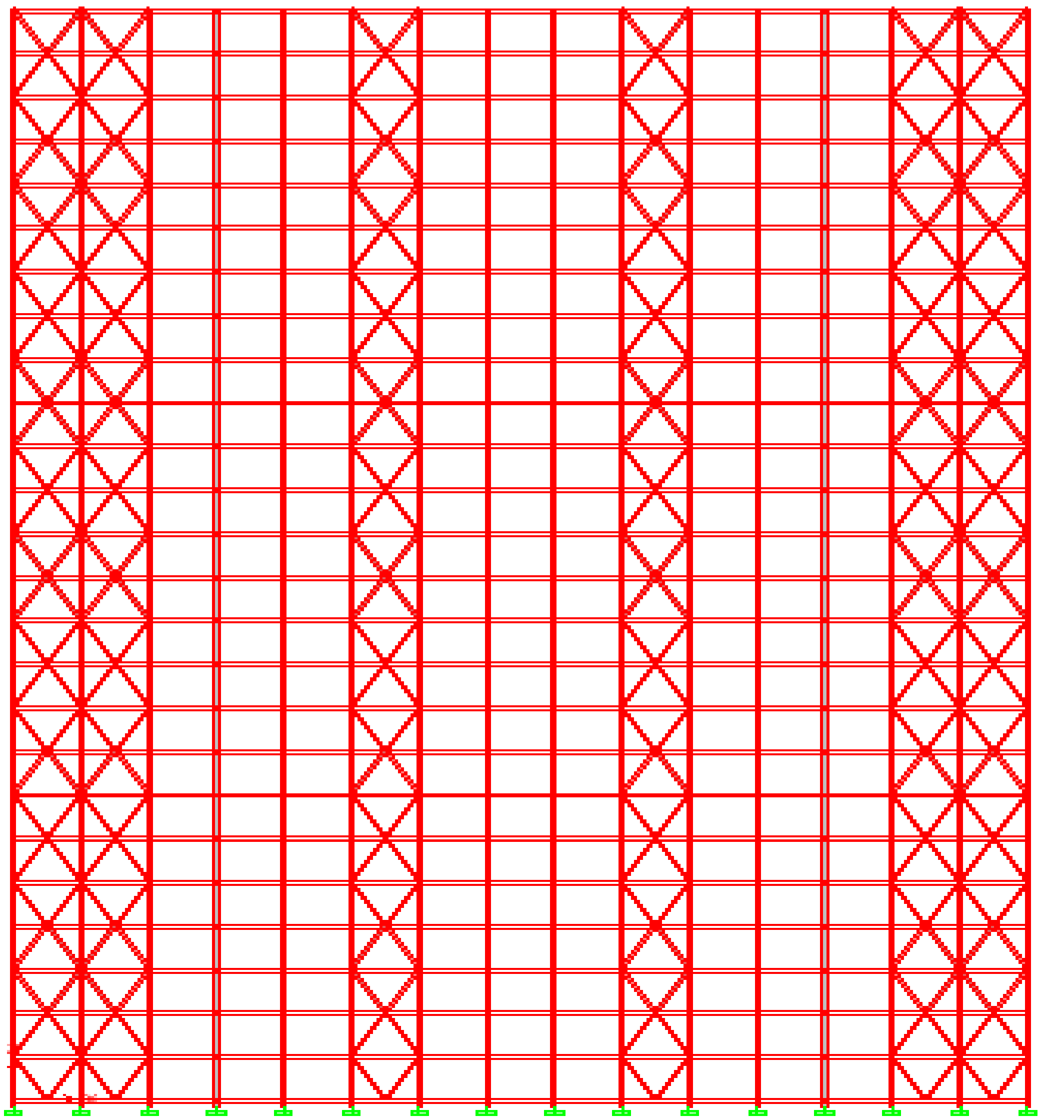


Figure 2.9- Combination of v and inverted v (chevron) bracing model 2

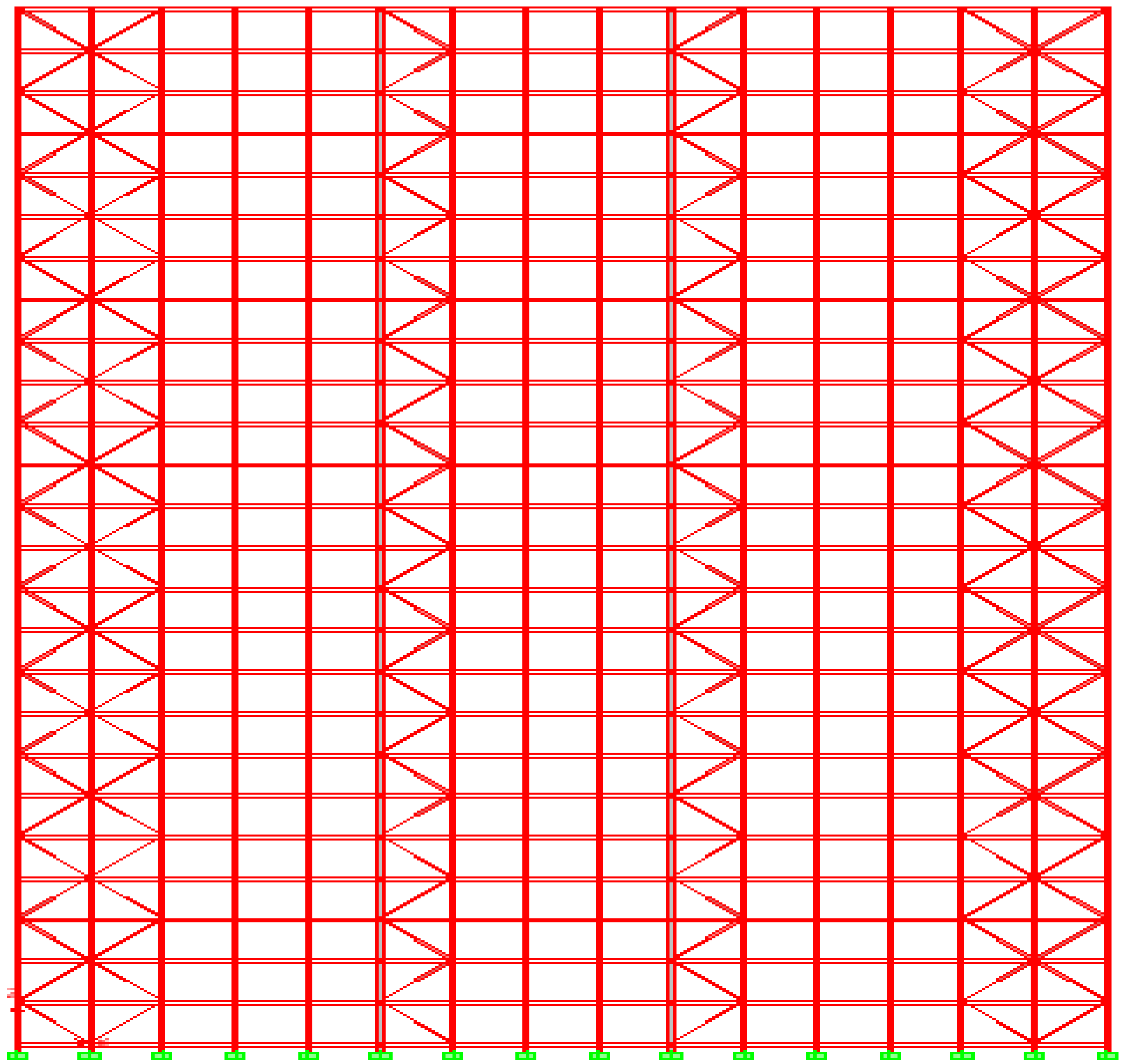


Figure 3.0- Concentric single-diagonal, alternate direction of bracing (Model 3)

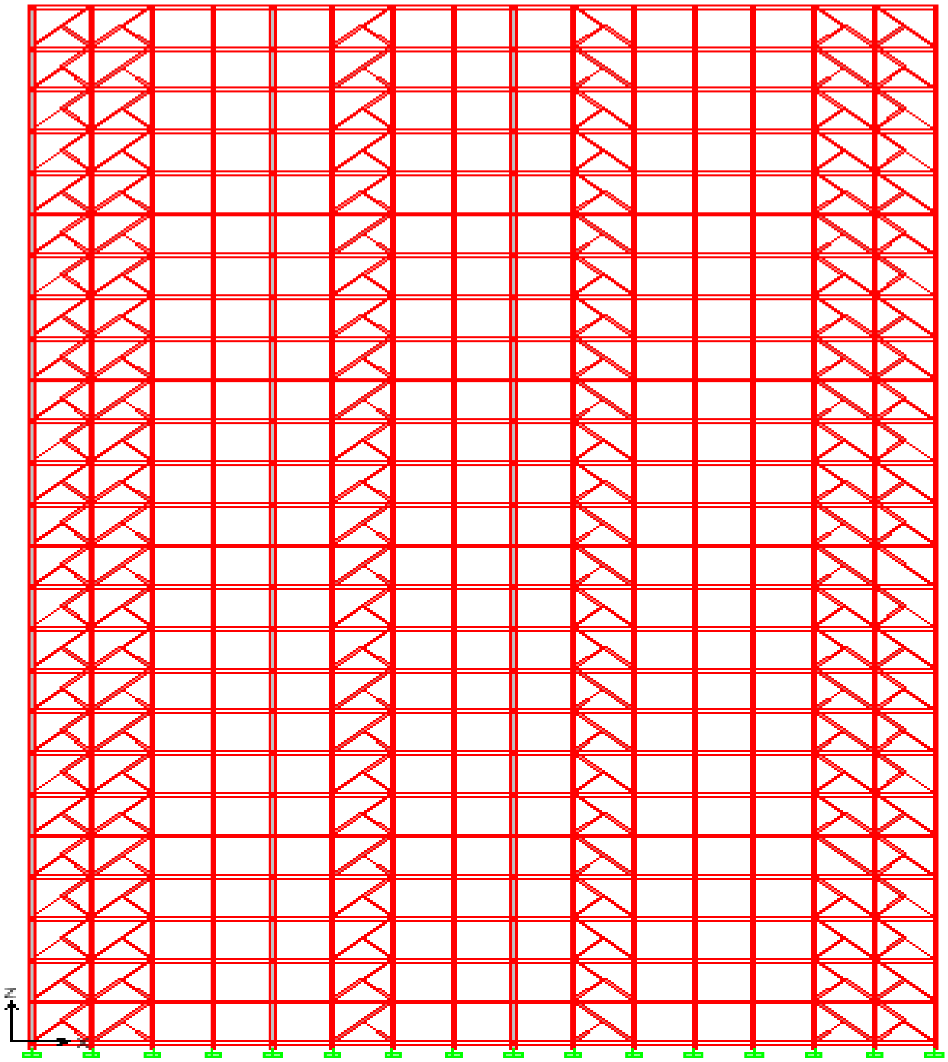


Figure 3.1- Concentric knee bracing system (Model 4)

4.3. Models of eccentric bracing systems (model 5, 6, 7, and 8)

For seismic applications the braces are designed such that they do not buckle under extreme loading conditions. This basic requirement can be assured since the ultimate capacity link can be accurately estimated, and an EBF is so proportioned that under severe loadings the major inelastic activity takes place in the ink. In this manner links provide the fuses necessary to prevent buckling of the braces. As to the efficiency of eccentric bracing for augmenting the elastic stiffness of a frame, it is instructive to compare the behavior of an EBF with a moment-resisting frame and a concentrically braced frame. For this purpose consider the variation as a function of the link length e. For e=L one has an MRF and the relative frame stiffness is at a minimum. For $e/L > 0.5$ little benefit is gained from the bracing. However, as the length of the link decreases, a rapid increase in elastic frame.

During the modeling of the eccentric bracing systems blows, the value of link (eccentricity),

e =1.34m is assumed for each, except for v bracing in which e = L/2=2 taking into account that the building should be stable. The following relation is applicable to determine the limit of eccentricity which is $e/L \leq 0.5$ where L is the bay length. There for, since the link length assumed for model 6, 7 and 8 is1.34m in which e/L= 1.34/4 = 0.335 less than 0.5 the lateral stiffness gained from bracing is high. Whereas the length of short link segment is 2m for eccentric v brace of model 5 in which e= 2/4 =0.5 is equal to 0.5.

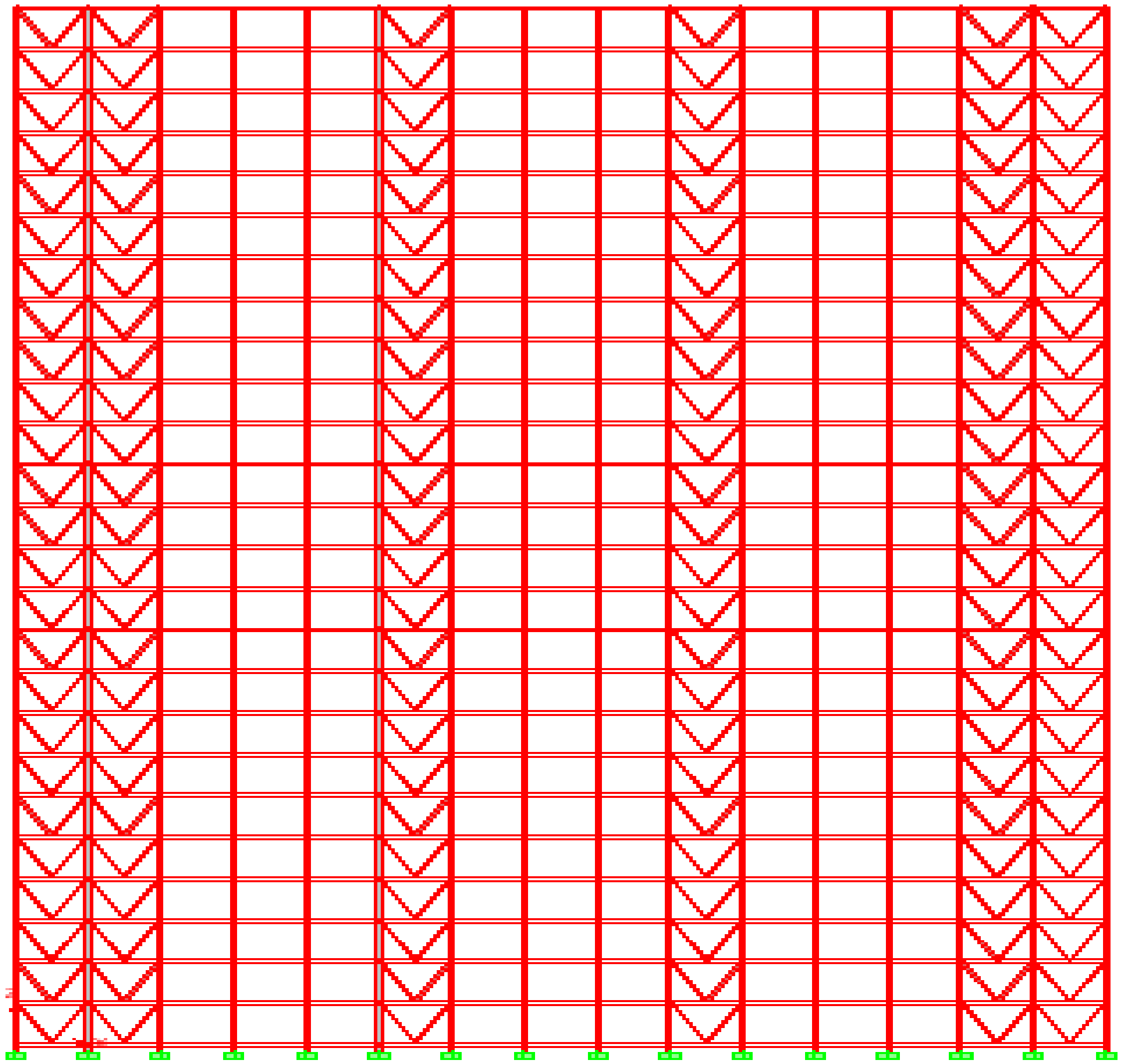


Figure 3.2- Eccentric v bracing system type one (Model 5)

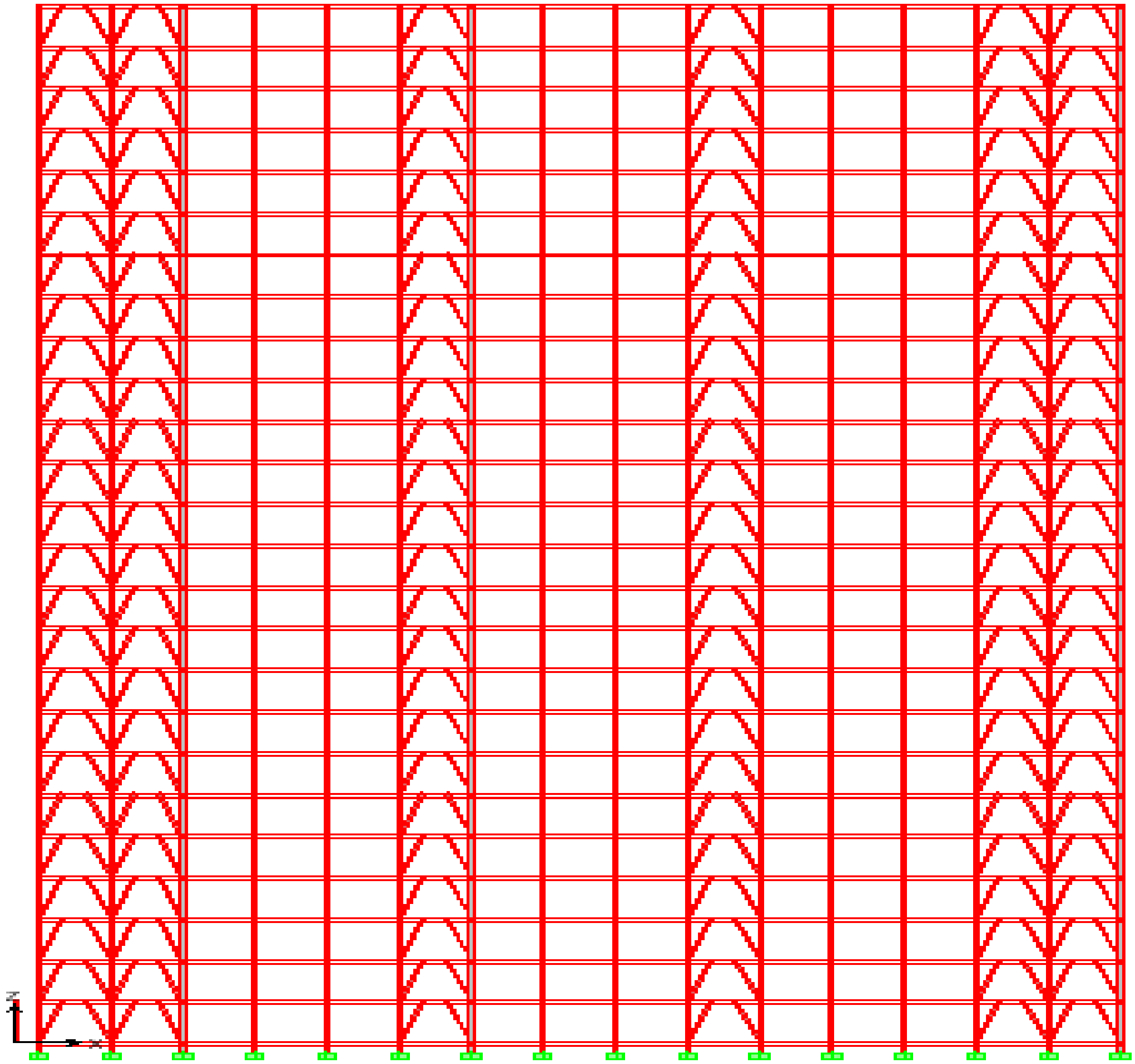


Figure 3.3- Eccentric bracing system type two (Model 6)

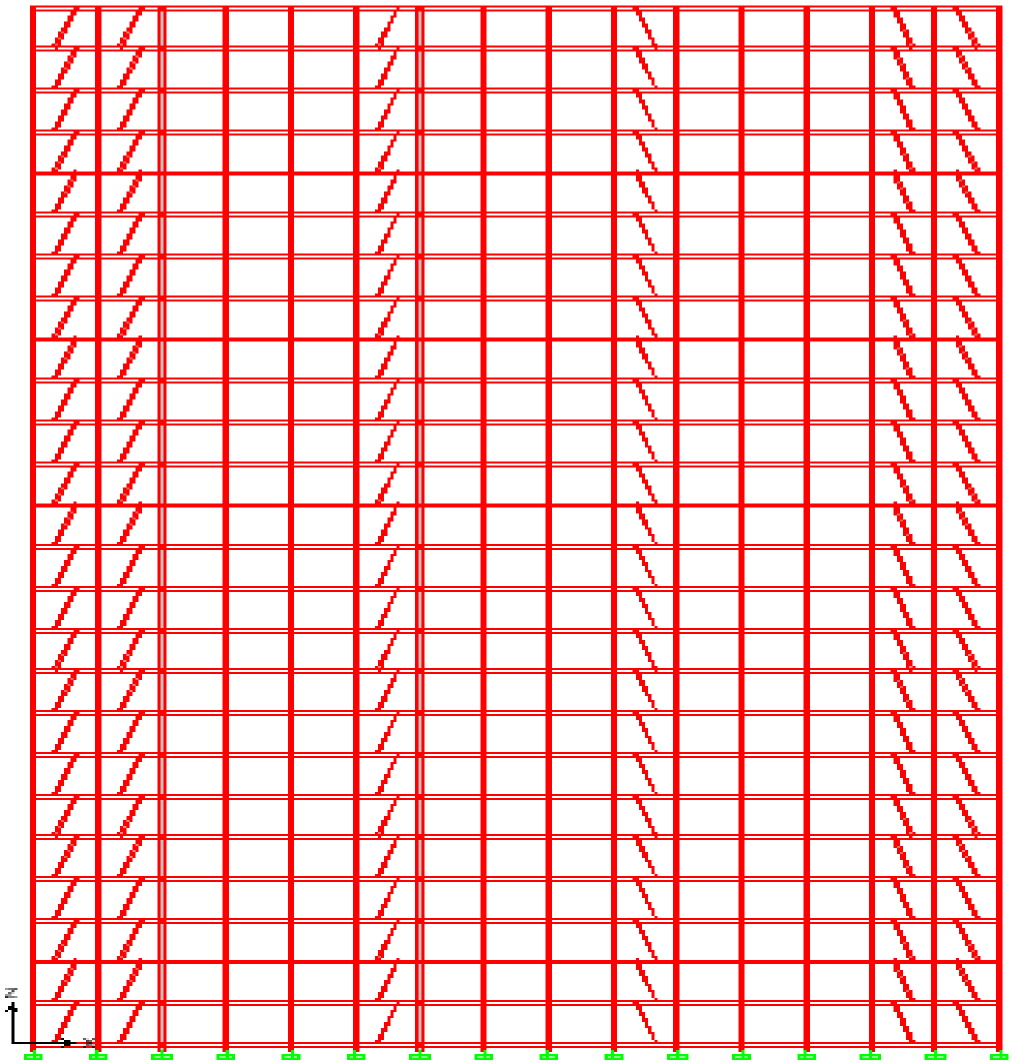


Figure 3.4- Eccentric bracing system, type three (Model 7)

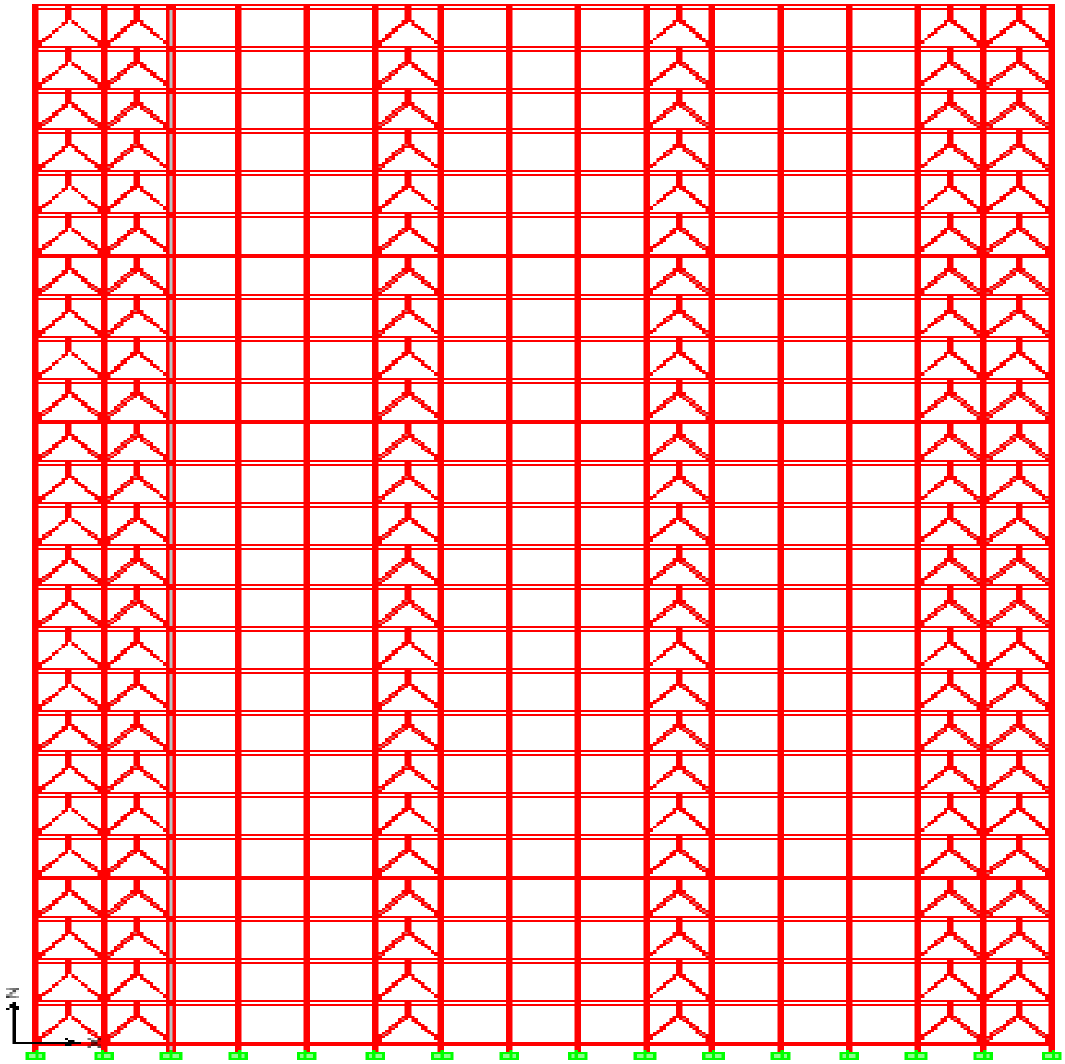


Figure 3.5- Eccentric bracing system type four (Model 8)

4.4. Analysis Result

4.5.1 Analysis Result of irregular twenty-five storey steel building.

This steel building frame is subjected to the previously specified loads and the corresponding lateral displacement and story drift at each storey level for the particular bracing types (X-bracing, combination of V and inverted V bracing), diagonal bracing, knee bracing- bracing and other three types of eccentric bracings) observed. When the earthquake force applied to the building in the X-direction some parametric result was found and presented in subsequent section shown below. The lateral displacement values of the frame are taken from the periphery of the building on the axis at which bracings are located.

Table 1.9 Lateral displacement and drift result for un-braced model at point object 16(EQX)

STOREY	DISP– X(mm)	DISP- Y(mm)	DRIFT - X	DRIFT -Y
Storey 25	195.473	-27.981	0.00195473	-0.00027981
Storey 24	194.086	-27.88	0.002021729	-0.000290417
Storey 23	191.96	-27.676	0.002086522	-0.000300826
Storey 22	189.043	-27.376	0.002148216	-0.000311091
Storey 21	185.38	-26.981	0.002206905	-0.000321202
Storey 20	181.005	-26.492	0.002262563	-0.00033115
Storey 19	175.956	-25.908	0.002315211	-0.000340895
Storey 18	170.266	-25.23	0.002364806	-0.000350417
Storey 17	163.971	-24.458	0.002411338	-0.000359676
Storey 16	157.106	-23.593	0.002454781	-0.000368641
Storey 15	149.708	-22.637	0.002495133	-0.000377283
Storey 14	141.811	-21.592	0.002532339	-0.000385571
Storey 13	133.453	-20.46	0.002566404	-0.000393462
Storey 12	124.669	-19.245	0.002597271	-0.000400938
Storey 11	115.496	-17.95	0.002624909	-0.000407955
Storey 10	105.972	-16.575	0.0026493	-0.000414375
Storey 9	96.131	-15.118	0.002670306	-0.000419944
Storey 8	85.946	-13.621	0.002685813	-0.000425656
Storey 7	75.946	-12.054	0.002712357	-0.0004305
Storey 6	64.896	-10.43	0.002704	-0.000434583
Storey 5	54.111	-8.757	0.00270555	-0.00043785
Storey 4	43.205	-7.042	0.002700313	-0.000440125
Storey 3	32.218	-5.293	0.002684833	-0.000441083
Storey 2	21.192	-3.519	0.002649	-0.000439875
Storey 1	10.199	-1.733	0.00254975	-0.00043325

Table 2.0 Lateral displacement and drift index for X braced (model 1) at point object 16(EQX)

STOREY	DISP – X(mm)	DISP - Y(mm)	DRIFT - X	DRIFT - Y
Storey 25	104.949	-8.935	0.00104949	-0.00008935
Storey 24	102.162	-8.841	0.001064188	-9.20938E-05
Storey 23	99.301	-8.652	0.001079359	-9.40435E-05
Storey 22	96.282	-8.419	0.001094114	-9.56705E-05
Storey 21	93.069	-8.161	0.001107964	-9.71548E-05
Storey 20	89.639	-7.879	0.001120488	-9.84875E-05
Storey 19	85.977	-7.576	0.001131276	-9.96842E-05
Storey 18	82.076	-7.254	0.001139944	-0.00010075
Storey 17	77.933	-6.91	0.001146074	-0.000101618
Storey 16	73.55	-6.546	0.001149219	-0.000102281
Storey 15	68.938	-6.163	0.001148967	-0.000102717
Storey 14	64.111	-5.76	0.001144839	-0.000102857
Storey 13	59.088	-5.34	0.001136308	-0.000102692
Storey 12	53.898	-4.906	0.001122875	-0.000102208
Storey 11	48.575	-4.458	0.001103977	-0.000101318
Storey 10	43.161	-4.003	0.001079025	-0.000100075
Storey 9	37.707	-3.542	0.001047417	-9.83889E-05
Storey 8	32.272	-3.082	0.0010085	-9.63125E-05
Storey 7	26.926	-2.628	0.000961643	-9.38571E-05
Storey 6	21.749	-2.186	0.000906208	-9.10833E-05
Storey 5	16.835	-1.762	0.00084175	-0.0000881
Storey 4	12.289	-1.363	0.000768063	-8.51875E-05
Storey 3	8.236	-0.996	0.000686333	-0.000083
Storey 2	4.819	-0.668	0.000602375	-0.0000835
Storey 1	2.214	-0.387	0.0005535	-0.00009675

Table 2.1 Lateral displacement and drift index for combination of V and inverted V braced (model 2)

STOREY	DISP – X(mm)	DISP - Y(mm)	DRIFT - X	DRIFT - Y
Storey 25	150.71	-4.026	0.0015071	-0.00004026
Storey 24	147.317	-3.656	0.001534552	-3.80833E-05
Storey 23	143.66	-3.369	0.001561522	-3.66196E-05
Storey 22	139.841	-3.071	0.001589102	-3.48977E-05
Storey 21	135.602	-2.804	0.00161431	-3.3381E-05
Storey 20	131.878	-2.056	0.001648475	-0.0000257
Storey 19	126.271	-2.248	0.001661461	-2.95789E-05
Storey 18	121.173	-1.953	0.001682958	-0.000027125
Storey 17	115.534	-1.708	0.001699029	-2.51176E-05
Storey 16	109.716	-1.414	0.001714313	-2.20938E-05
Storey 15	103.371	-1.173	0.00172285	-0.00001955
Storey 14	96.843	-0.92	0.001729339	-1.64286E-05
Storey 13	89.854	-0.718	0.001727962	-1.38077E-05
Storey 12	82.688	-0.504	0.001722667	-0.0000105
Storey 11	75.185	-0.336	0.00170875	-7.63636E-06
Storey 10	67.521	-0.174	0.001688025	-0.00000435
Storey 9	59.719	-0.044	0.001658861	-1.22222E-06
Storey 8	51.781	-0.052	0.001618156	-0.000001625
Storey 7	44.033	-0.145	0.001572607	-5.17857E-06
Storey 6	36.126	-0.167	0.00150525	-6.95833E-06
Storey 5	28.829	-0.231	0.00144145	-0.00001155
Storey 4	21.489	-0.017	0.001343063	-1.0625E-06
Storey 3	15.299	-0.222	0.001274917	-0.0000185
Storey 2	9.151	-0.083	0.001143875	-0.000010375
Storey 1	4.897	-0.08	0.00122425	-0.00002

Table 2.2 Lateral displacement and drift index of diagonal braced (model 3) at point object 16

STOREY	DISP – X(mm)	DISP - Y(mm)	DRIFT - X	DRIFT – Y
Storey 25	55.206	-4.309	0.00055206	-0.00004309
Storey 24	53.896	-4.266	0.000561417	-4.44375E-05
Storey 23	52,492	-4.186	0.570565217	-0.0000455
Storey 22	51.012	-4.099	0.000579682	-4.65795E-05
Storey 21	49.393	-3.994	0.000588012	-4.75476E-05
Storey 20	47.69	-3.887	0.000596125	-4.85875E-05
Storey 19	45.826	-3.759	0.000602974	-4.94605E-05
Storey 18	43.876	-3.629	0.000609389	-5.04028E-05
Storey 17	41.759	-3.478	0.000614103	-5.11471E-05
Storey 16	39.557	-3.324	0.000618078	-5.19375E-05
Storey 15	37.198	-3.151	0.000619967	-5.25167E-05
Storey 14	34.759	-2.972	0.000620696	-5.30714E-05
Storey 13	32.187	-2.778	0.000618981	-5.34231E-05
Storey 12	29.546	-2.575	0.000615542	-5.36458E-05
Storey 11	26.813	-2.364	0.000609386	-5.37273E-05
Storey 10	24.028	-2.14	0.0006007	-0.0000535
Storey 9	21.214	-1.918	0.000589278	-5.32778E-05
Storey 8	18.372	-1.678	0.000574125	-5.24375E-05
Storey 7	15.586	-1.453	0.000556643	-5.18929E-05
Storey 6	12.807	-1.205	0.000533625	-5.02083E-05
Storey 5	10.2	-0.99	0.00051	-0.0000495
Storey 4	7.645	-0.746	0.000477813	-0.000046625
Storey 3	5.415	-0.559	0.00045125	-4.65833E-05
Storey 2	3.298	-0.338	0.00041225	-0.00004225
Storey 1	1.703	-0.197	0.00042575	-0.00004925

Table 2.3 Lateral displacement and drift index for knee braced (model 4) at point object 1

STOREY	DISP – X(mm)	DISP – Y(mm)	DRIFT - X	DRIFT –Y
Storey 25	99.484	-2.927	0.00099484	-0.00002927
Storey 24	97.178	-3.0622	0.001012271	-3.18979E-05
Storey 23	94.751	-3.133	0.001029902	-3.40543E-05
Storey 22	92.146	-3.185	0.001047114	-3.61932E-05
Storey 21	89.338	-3.222	0.001063548	-3.83571E-05
Storey 20	86.314	-3.246	0.001078925	-0.000040575
Storey 19	83.061	-3.256	0.001092908	-4.28421E-05
Storey 18	79.572	-3.252	0.001105167	-4.51667E-05
Storey 17	75.848	-3.232	0.001115412	-4.75294E-05
Storey 16	71.889	-3.196	0.001123266	-4.99375E-05
Storey 15	67.703	-3.143	0.001128383	-5.23833E-05
Storey 14	63.301	-3.072	0.001130375	-5.48571E-05
Story 13	58.698	-2.981	0.001128808	-5.73269E-05
Storey 12	53.916	-2.869	0.00112325	-5.97708E-05
Storey 11	48.981	-2.737	0.001113205	-6.22045E-05
Storey 10	43.926	-2.582	0.00109815	-0.00006455
Storey 9	38.791	-2.406	0.001077528	-6.68333E-05
Storey 8	33.625	-2.206	0.001050781	-6.89375E-05
Storey 7	28.485	-1.984	0.001017321	-7.08571E-05
Storey 6	23.438	-1.741	0.000976583	-7.25417E-05
Storey 5	18.562	-1.479	0.0009281	-0.00007395
Storey 4	13.951	-1.2	0.000871938	-0.000075
Storey 3	9.713	-0.904	0.000809417	-7.53333E-05
Storey 2	5.977	-0.616	0.000747125	-0.000077
Storey1	2.88	-0.332	0.00072	-0.000083

Table 2.4 Lateral displacement and drift index of v braced (model 5) at point object 16(EQX)

STOREY	DISP– X(mm)	DISP - Y(mm)	DRIFT - X	DRIFT –Y
Storey 25	63.415	-12.409	0.00063415	-0.00012409
Storey 24	62.13	-12.056	0.000647188	-0.000125583
Storey 23	60.763	-11.653	0.000660467	-0.000126663
Storey 22	59.284	-11.226	0.000673682	-0.000127568
Storey 21	57.674	-10.774	0.000686595	-0.000128262
Storey 20	55.925	-10.3	0.000699063	-0.00012875
Storey 19	54.028	-9.801	0.000710895	-0.000128961
Storey 18	51.975	-9.278	0.000721875	-0.000128861
Storey 17	49.766	-8.728	0.000731853	-0.000128353
Storey 16	47.399	-8.154	0.000740609	-0.000127406
Storey 15	44.876	-7.555	0.000747933	-0.000125917
Storey 14	42.202	-6.931	0.000753607	-0.000123768
Storey 13	39.384	-6.287	0.000757385	-0.000120904
Storey 12	36.433	-5.624	0.000759021	-0.000117167
Storey 11	33.362	-4.945	0.000758227	-0.000112386
Storey 10	30.191	-4.257	0.000754775	-0.000106425
Storey 9	26.942	-3.564	0.000748389	-0.000099
Storey 8	23.644	-2.872	0.000738875	-0.00008975
Storey 7	20.331	-2.189	0.000726107	-7.81786E-05
Storey 6	17.045	-1.524	0.000710208	-0.0000635
Storey 5	13.832	-0.885	0.0006916	-0.00004425
Storey 4	10.751	-0.2778	0.000671938	-1.73625E-05
Storey 3	7.866	-0.254	0.0006555	-2.11667E-05
Storey 2	5.237	-0.21	0.000654625	-0.00002625
Storey 1	2.829	-0.123	0.00070725	-0.00003075

Table 2.5 Lateral displacement and drift index of Eccentric bracing system type four (Model 6)

STOREY	DISP– X(mm)	DISP – Y(mm)	DRIFT - X	DRIFT –Y
Storey 25	159.012	-15.441	0.00159012	-0.00015441
Storey 24	157.344	-15.218	0.001639	-0.000158521
Storey 23	155.295	-14.957	0.001687989	-0.000162576
Storey 22	152.87	-14.666	0.001737159	-0.000166659
Storey 21	150.051	-14.344	0.001786321	-0.000170762
Storey 20	146.819	-13.993	0.001835238	-0.000174913
Storey 19	143.156	-13.608	0.001883632	-0.000179053
Storey 18	139.054	-13.189	0.001931306	-0.000183181
Storey 17	134.493	-12.732	0.001977838	-0.000187235
Storey 16	129.463	-12.237	0.002022859	-0.000191203
Storey 15	123.956	-11.699	0.002065933	-0.000194983
Storey 14	117.966	-11.12	0.002106536	-0.000198571
Storey 13	111.491	-10.496	0.002144058	-0.000201846
Storey 12	104.532	-9.814	0.00217775	-0.000204458
Storey 11	97.095	-9.101	0.002206705	-0.000206841
Storey 10	89.195	-8.345	0.002229875	-0.000208625
Storey 9	80.856	-7.546	0.002246	-0.000209611
Storey 8	72.114	-6.709	0.002253563	-0.000209656
Storey 7	63.02	-5.839	0.002250714	-0.000208536
Storey 6	53.644	-4.942	0.002235167	-0.000205917
Storey 5	44.08	-4.03	0.002204	-0.0002015
Storey 4	34.456	-3.115	0.0021535	-0.000194688
Storey 3	24.938	-2.217	0.002078167	-0.00018475
Storey 2	15.745	-1.361	0.001968125	-0.000170125
Storey 1	7.192	-0.583	0.001798	-0.00014575

Table 2.6 Lateral displacement and drift of Eccentric bracing system type four (Model 7)

STOREY	DISP – X(mm)	DISP - Y(mm	DRIFT - X	DRIFT – y
Storey 25	181.603	-32.317	0.00181603	-0.00032317
Storey 24	180.262	-32.012	0.001877729	-0.000333458
Storey 23	178.423	-31.65	0.00193938	-0.000344022
Storey 22	176.15	-31.243	0.002001705	-0.000355034
Storey 21	173.454	-30.794	0.002064929	-0.000366595
Storey 20	170.321	-30.299	0.002129013	-0.000378738
Storey 19	166.735	-29.754	0.002193882	-0.0003915
Storey 18	162.685	-29.159	0.002259514	-0.000404986
Storey 17	158.154	-28.504	0.002325794	-0.000419176
Storey 16	153.124	-27.781	0.002392563	-0.000434078
Storey 15	147.582	-26.983	0.0024597	-0.000449717
Storey 14	141.515	-26.104	0.002527054	-0.000466143
Storey 13	134.909	-25.135	0.002594404	-0.000483365
Storey 12	127.755	-24.068	0.002661563	-0.000501417
Storey 11	120.043	-22.892	0.00272825	-0.000520273
Storey 10	111.765	-21.593	0.002794125	-0.000539825
Storey 9	102.923	-20.179	0.002858972	-0.000560528
Storey 8	93.515	-18.603	0.002922344	-0.000581344
Storey 7	83.554	-16.901	0.002984071	-0.000603607
Storey 6	73.059	-15.044	0.003044125	-0.000626833
Storey 5	62.059	-13.021	0.00310295	-0.00065105
Storey 4	50.592	-10.836	0.003162	-0.00067725
Storey 3	38.694	-8.485	0.0032245	-0.000707083
Storey 2	26.29	-5.863	0.00328625	-0.000732875
Storey 1	13.06	-2.726	0.003265	-0.0006815

Table 2.7 Lateral displacement and drift of Eccentric bracing system type four (Model 8)

STOREY	DISP – X(mm)	DISP - Y(mm)	DRIFT - X	DRIFT – X
Storey 25	170.034	-72.553	0.00170034	-0.00072553
Storey 24	168.947	-72.047	0.001759865	-0.00075049
Storey 23	167.176	-71.405	0.00181713	-0.000776141
Storey 22	164.996	-70.686	0.001874955	-0.00080325
Storey 21	162.391	-69.893	0.001933226	-0.00083206
Storey 20	159.347	-69.018	0.001991838	-0.000862725
Storey 19	155.849	-68.051	0.002050645	-0.000895408
Storey 18	151.885	-66.976	0.002109514	-0.000930222
Storey 17	147.443	-65.783	0.002168279	-0.000967397
Storey 16	142.51	-64.453	0.002226719	-0.001007078
Storey 15	137.074	-62.968	0.002284567	-0.001049467
Storey 14	131.127	-61.306	0.002341554	-0.00109475
Storey 13	124.658	-59.439	0.002397269	-0.001143058
Storey 12	117.66	-57.328	0.00245125	-0.001194333
Storey 11	110.131	-54.958	0.002502977	-0.001249045
Storey 10	102.078	-52.4	0.00255195	-0.00131
Storey 9	93.501	-49.441	0.00259725	-0.001373361
Storey 8	84.416	-46.106	0.002638	-0.001440813
Storey 7	74.851	-42.32	0.00267325	-0.001511429
Storey 6	64.837	-38.016	0.002701542	-0.001584
Storey 5	54.42	-33.127	0.002721	-0.00165635
Storey 4	43.589	-27.614	0.002724313	-0.001725875
Storey 3	32.472	-20.7	0.002706	-0.001725
Storey 2	21.339	-13.494	0.002667375	-0.00168675
Storey 1	10.33	-5.678	0.0025825	-0.0014195

4.6 COMPARISONS AND DISCUSSION OF LATERAL DISPLACEMENTS

4.6.1 Analysis Result

The previously analyzed building for each bracing systems of the same storey is plotted using excel spreadsheet for comparison purpose and the corresponding result is tabulated as shown below for their corresponding analysis results.

Table 2.8 Maximum nodal displacement at the top storey in X direction**.**

Model	Node displacement (mm)
Reference model	**195.473**
Model 1	104.949
Model 2	150.71
Model 3	55.206
Model 4	99.484
Model 5	63.415
Model 6	159.012
Model 7	181.603
Model 8	170.034

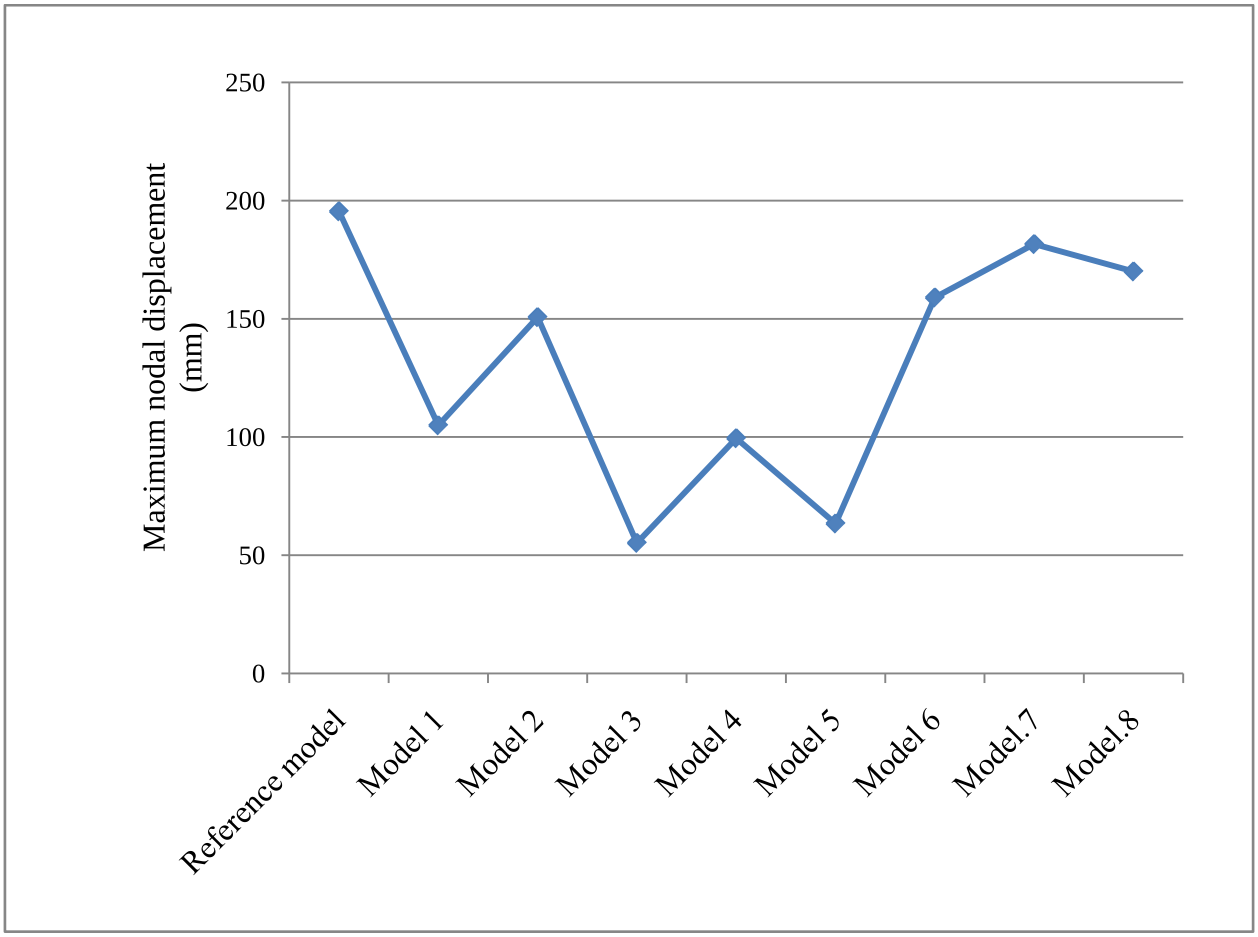


Fig 3.6- Maximum nodal displacement for different models in X direction.

Table 2.9 Reduction in drift index percentage for various models in comparison with un-braced model along X direction in Zone IV.

Model	DISP-X (mm)	DRIFT - X	Percentage reduction (%)
Reference model	**195.473**	**0.00195473**	**-------**
Model 1	104.949	0.00104949	46.310
Model 2	150.71	0.0015071	22.899
Model 3	55.206	0.00055206	71.757
Model 4	99.484	0.00099484	49.106
Model 5	63.415	0.00063415	67.558
Model 6	159.012	0.00159012	18.652
Model.7	181.603	0.00181603	7.095
Model.8	170.034	0.00170034	13.014

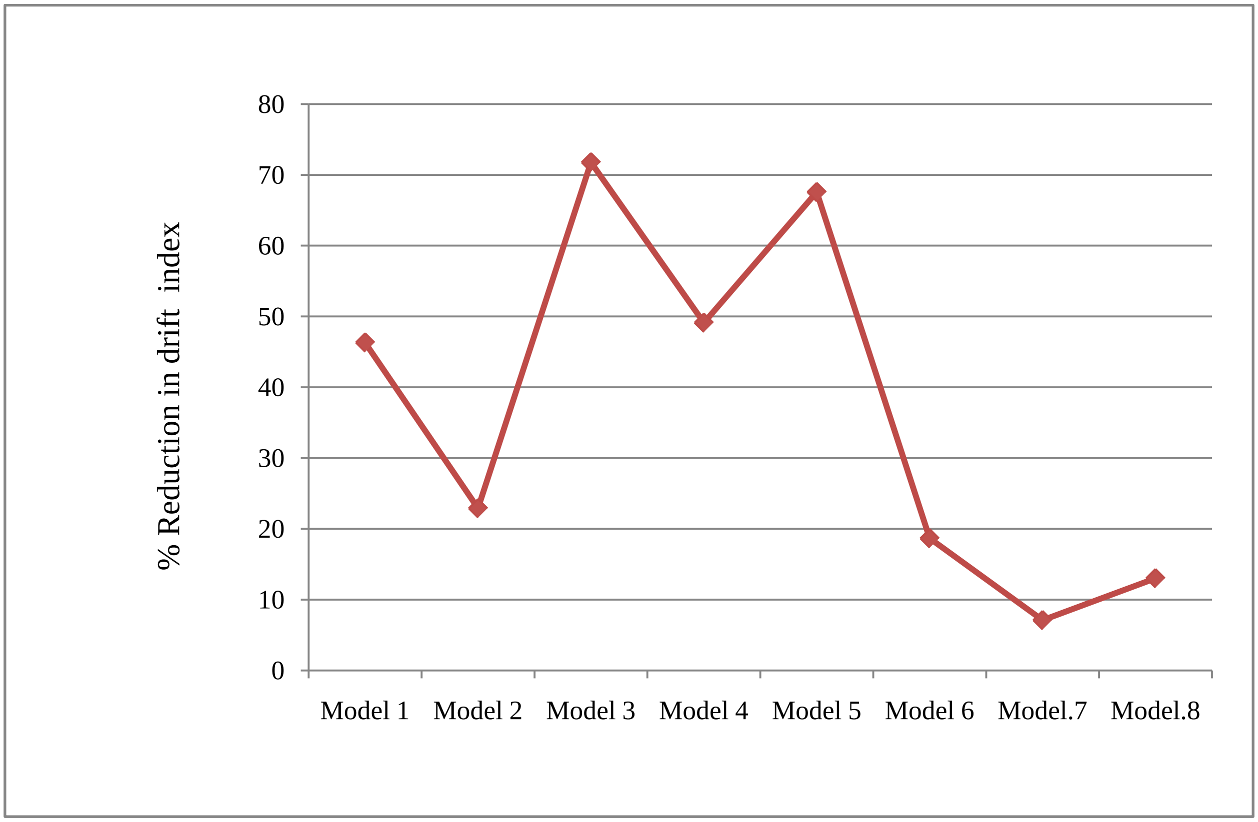


Fig 3.7- Reduction in drift index percentage versus various models considered along X direction

Table 3.0 Lateral displacement values of each model using the braces.

Storey	**Concentric bracings**				**Eccentric bracings**			
	X brace	Comb	diagonal	Knee	V type	Model 6	Model 7	Model 8
St.25	104.949	150.71	55.206	99.484	63.415	159.012	181.603	170.034
St.24	102.162	147.317	53.896	97.178	62.13	157.344	180.262	168.947
St.23	99.301	143.66	52,492	94.751	60.763	155.295	178.423	167.176
St.22	96.282	139.841	51.012	92.146	59.284	152.87	176.15	164.996
St.21	93.069	135.602	49.393	89.338	57.674	150.051	173.454	162.391
St.20	89.639	131.878	47.69	86.314	55.925	146.819	170.321	159.347
St.19	85.977	126.271	45.826	83.061	54.028	143.156	166.735	155.849
St.18	82.076	121.173	43.876	79.572	51.975	139..054	162.685	151.885
St.17	77.933	115.534	41.759	75.848	49.766	134.493	158.154	147.443
St.16	73.55	109.716	39.557	71.889	47.399	129.463	153.124	142.51
St.15	68.938	103.371	37.198	67.703	44.876	123.956	147.582	137.074
St.14	64.111	96.843	34.759	63.301	42.202	117.966	141.515	131.127
St.13	59.088	89.854	32.187	58.698	39.384	111.491	134.909	124.658
St.12	53.898	82.688	29.546	53.916	36.433	104.532	127.755	117.66
St.11	48.575	75.185	26.813	48..981	33.362	97.095	120.043	110.131
St.10	43.161	67.521	24.028	43.926	30.191	89.195	111.765	102.078
St.9	37.707	59.719	21.214	38.791	26.942	80.856	102.923	93.501
St.8	32.272	51.781	18.372	33.625	23.644	72.114	93.515	84.416
St.7	26.926	44.033	15.586	28.485	20.331	63.02	83.554	74.851
St.6	21.749	36.126	12.807	23.438	17.045	53.644	73.059	64.837
St.5	16.835	28.829	10.2	18.562	13.832	44.08	62.059	54.42
St.4	12.289	21.489	7.645	13.951	10.751	34..456	50.592	43.589
St.3	8.236	15.299	5.415	9.713	7.866	24.938	38.694	32.472
St.2	4.819	9.151	3.298	5.977	5.237	15.745	26.29	21.339
St.1	2.214	4.897	1.703	2.88	2.829	7.192	13.06	10.33

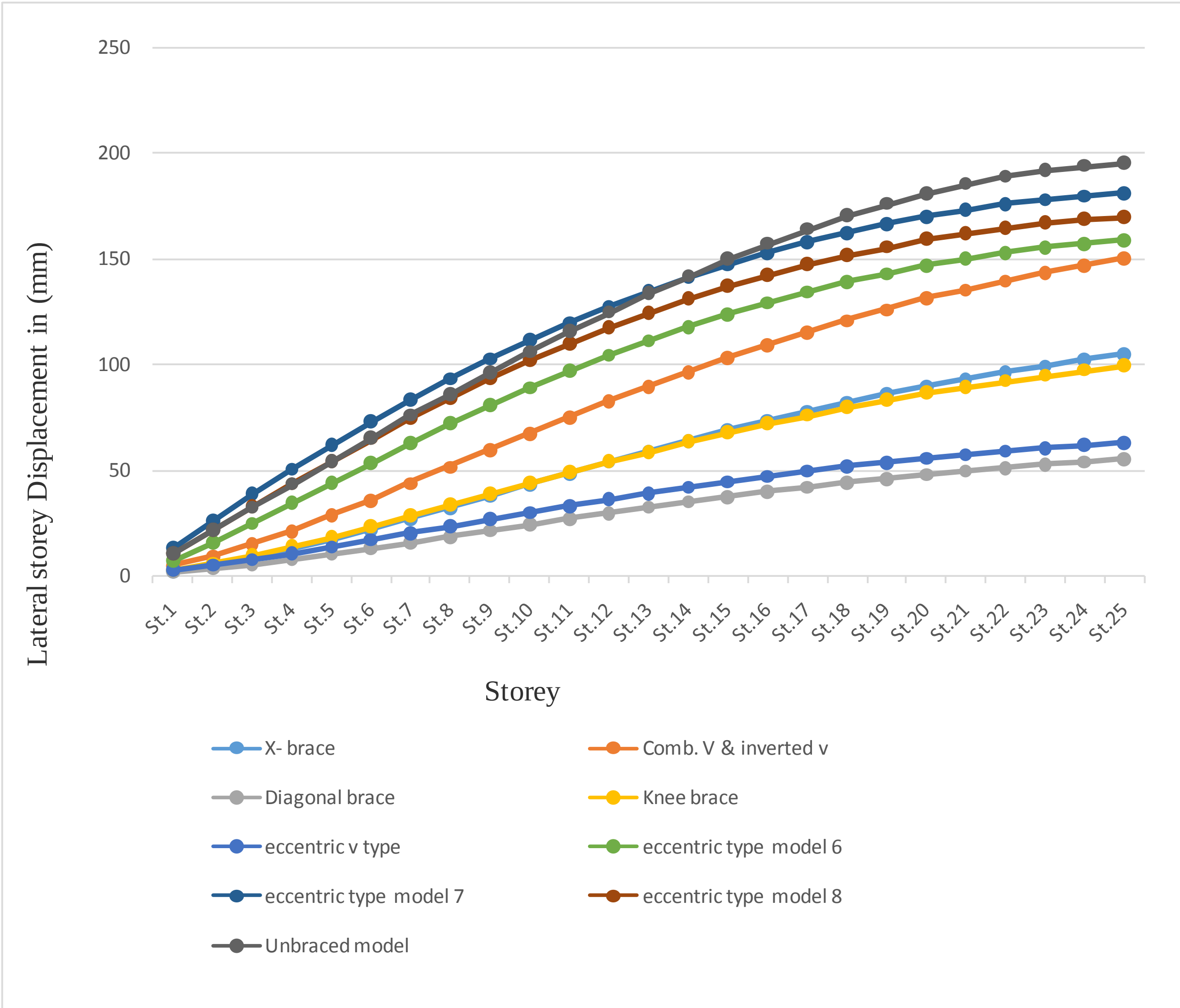


Figure 3.8- Plot of lateral displacement values of each bracing types for twenty-five storey building

In addition to lateral displacement values of the building, it is also very important to check its storey drift values. Story drift is the difference in horizontal deflection at the top and bottom of any story or it is the ratio of maximum displacement at the top of the building to the height of the building and the corresponding results of each storey are tabulated as following.

Table 3.1 Lateral Storey drift index in the x- direction for the eight types of bracings.

Storey	**Concentric bracings**				**Eccentric bracings**			
	X brace	**Comb.**	**diagonal**	**Knee**	**V type**	**Model 6**	**Model 7**	**Model 8**
St.25	0.00105	0.00151	0.00055	0.00099	0.00063	0.00159	0.00182	0.0017
St.24	0.00106	0.00153	0.00056	0.00101	0.00065	0.00164	0.00188	0.00176
St.23	0.00108	0.00156	0.57057	0.00103	0.00066	0.00169	0.00194	0.00182
St.22	0.00109	0.00159	0.00058	0.00105	0.00067	0.00174	0.002	0.00187
St.21	0.00111	0.00161	0.00059	0.00106	0.00069	0.00179	0.00206	0.00193
St.20	0.00112	0.00165	0.0006	0.00108	0.0007	0.00184	0.00213	0.00199
St.19	0.00113	0.00166	0.0006	0.00109	0.00071	0.00188	0.00219	0.00205
St.18	0.00114	0.00168	0.00061	0.00111	0.00072	0.00193	0.00226	0.00211
St.17	0.00115	0.0017	0.00061	0.00112	0.00073	0.00198	0.00233	0.00217
St.16	0.00115	0.00171	0.00062	0.00112	0.00074	0.00202	0.00239	0.00223
St.15	0.00115	0.00172	0.00062	0.00113	0.00075	0.00207	0.00246	0.00228
St.14	0.00114	0.00173	0.00062	0.00113	0.00075	0.00211	0.00253	0.00234
St.13	0.00114	0.00173	0.00062	0.00113	0.00076	0.00214	0.00259	0.0024
St.12	0.00112	0.00172	0.00062	0.00112	0.00076	0.00218	0.00266	0.00245
St.11	0.0011	0.00171	0.00061	0.00111	0.00076	0.00221	0.00273	0.0025
St.10	0.00108	0.00169	0.0006	0.0011	0.00075	0.00223	0.00279	0.00255
St.9	0.00105	0.00166	0.00059	0.00108	0.00075	0.00225	0.00286	0.0026
St.8	0.00101	0.00162	0.00057	0.00105	0.00074	0.00225	0.00292	0.00264
St.7	0.00096	0.00157	0.00056	0.00102	0.00073	0.00225	0.00298	0.00267
St.6	0.00091	0.00151	0.00053	0.00098	0.00071	0.00224	0.00304	0.0027
St.5	0.00084	0.00144	0.00051	0.00093	0.00069	0.0022	0.0031	0.00272
St.4	0.00077	0.00134	0.00048	0.00087	0.00067	0.00215	0.00316	0.00272
St.3	0.00069	0.00127	0.00045	0.00081	0.00066	0.00208	0.00322	0.00271
St.2	0.0006	0.00114	0.00041	0.00075	0.00065	0.00197	0.00329	0.00267
St.1	0.00055	0.00122	0.00043	0.00072	0.00071	0.0018	0.00327	0.00258

Table 3.2 Maximum axial force induced in the column for different bracing systems

Model number	**Axial force in (KN)**	**% increase**
Reference model	**1780.981**	------------
Model 1	3646.394	51.157
Model 2	3051.813	41.641
Model 3	2905.235	38.697
Model 4	3201.684	44.373
Model 5	3051.813	41.642
Model 6	2859.375	37.714
Model 7	2229.683	20.124
Model 8	2915.432	38.912

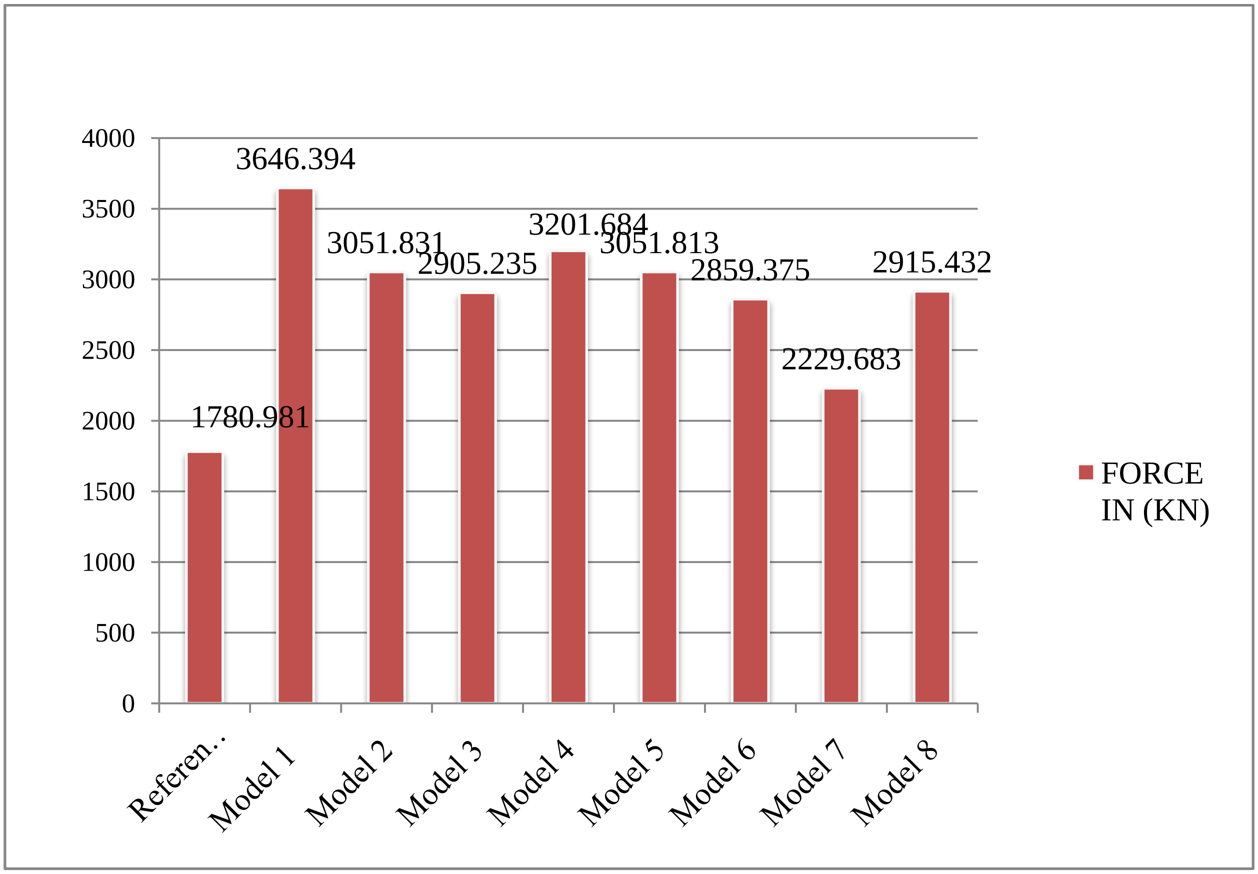


Fig 3.9- Variation of axial force on column for different bracing system.

Table 3.3 Maximum bending moment induced in the different bracing systems

Model number	**Bending moment (KN-m)**
Reference model	**127.912**
Model 1	148.781
Model 2	124.521
Model 3	118.540
Model 4	130.636
Model 5	124.521
Model 6	116.669
Model 7	90.948
Model 8	118.956

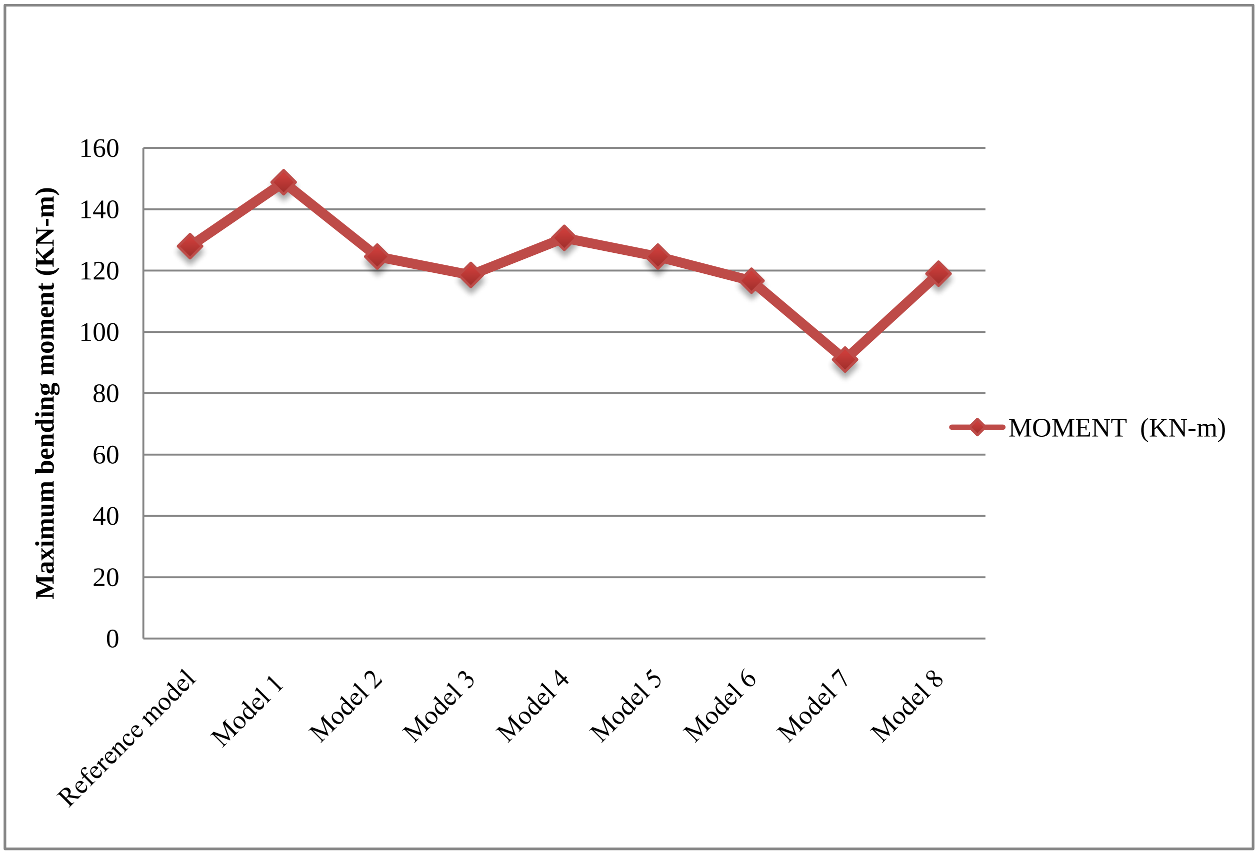


Fig 4.0- Maximum bending moment in column versus different bracing systems

Table 3.4 Quantity of structural steel braces used in the model for different bracing systems

Model number	Type of bracings	Weight of steel brace(KN)
Model 1	X bracing system	1885.60
Model 2	Combination of v and inverted v	1583.63
Model 3	Single diagonal	1000.84
Model 4	Knee bracing	1502.15
Model 5	Eccentric v bracing system	1490.52
Model 6	Eccentric type	1319.17
Model 7	Eccentric(/) type	745.76
Model 8	eccentric	1417.12

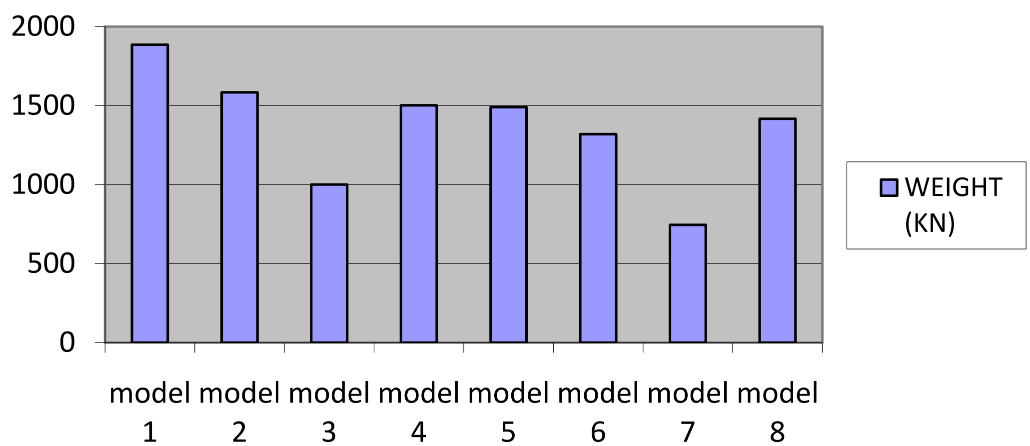


Figure 4.1 - Variation in quantity of steel brace for different bracing arrangement.

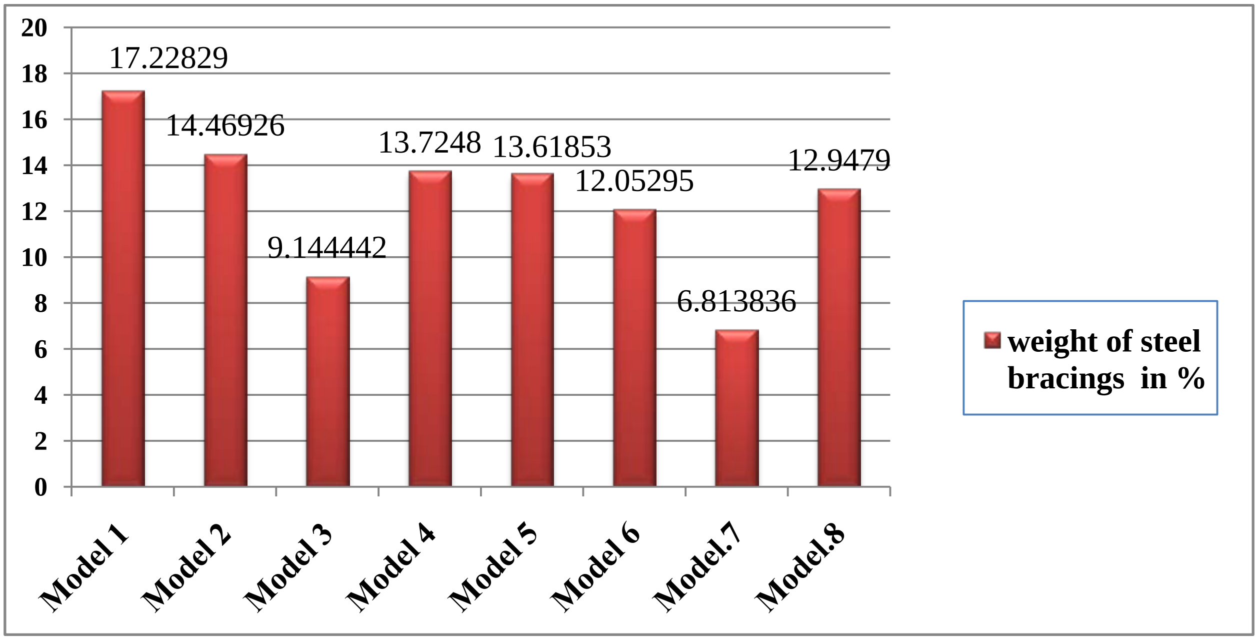


Figure 4.2 – Weight of bracings used in each model in percent from the total weight of bracings for all models.

4.4.1 Discussions on obtained results

The presence of bracing gives an advantage to increase the spacing between columns. From figure 3.8, the Plot of lateral displacement values of each bracing types, shows that:

- The lateral displacement values of knee bracing and X-bracing are exactly similar that they are overlapping with each other. They also have small displacement values compared to combination of V and inverted V bracing, and eccentric bracing types of (model 6, 7 &8). Eccentric bracings of (model 6, 7, 8) give higher lateral displacement values than the other. This shows that lateral stiffness capacity of eccentric type of (model 6, 7 & 8) bracings are lesser compared to other bracing types.
- From the comparison of the bracing systems; both single-diagonal, alternate direction bracing and eccentric type of V bracing have similar and smaller values of lateral displacement
- In Chevron bracing the lateral force is mainly resisted by the axial compression and tension effect of inclined bracing members. The advantage of this bracing is to minimize stress on the neighborhood beams and columns. While chevron bracing is used, the beam must be designed for an unbalanced load when the compression brace buckles. Often the resulting brace frame beam design weighing more
- For some bracing systems, the story drift values will increase from one story to the next storey then it reduces gradually for higher story levels.
- The storey drift values are checked and it is in the allowable range (as per Euro code 8 where cladding elements are rigidly attached to the structure, the SLS story drift is limited to 0.5% of storey height but this rise to 0.75% for rigidly attached ductile cladding). Compared to other bracing system, eccentric type (model 6) and eccentric type (model 7) bracings have higher values of story drift.
- From concentric braces studied; single diagonal- alternate direction brace (model 3) has small story drift value; as shown on figure 3.8. And also among eccentric braces, eccentric V type brace results small story drifts.
- The quantity of steel brace used for eccentric type model seven (7) is less than all other seven bracing systems where as the quantity of x bracing system is greater than all other bracing types investigated.
- The total weight of single diagonal alternate direction steel brace is greater than the total weight of eccentrically steel brace used for model seven by 2.331% but less than others.

CHAPTER5. CONCLUSIONS, RECOMMENDATIONS AND FUTURE SCOPE OF THE WORK

5.1 Conclusion

The following conclusions are after analyzing and observing analyses result of eight bracing types from concentric and eccentric bracing systems using ETABs soft ware relative to analyses results of the model without bracing.

1. Eccentric V type (model 5) and single – diagonal, alternate direction (model 3) have least nodal displacements with respect to storey height when compared to un-braced reference model. They also have lesser values of horizontal displacement compared to other bracings. This indicates that they can give higher resistance mechanism for the overall building structure**.**

2. Eccentric types of (model 6, 7 and 8) have lesser efficiency to resist lateral displacement because their nodal displacement is high relative to others. Thus it is better to avoid.

3. It is noticed that from the comparison of lateral displacement, the structural system that contain X bracing and knee bracing have almost equal values of horizontal displacement. The horizontal displacements of these bracings are higher than lateral displacements of single-diagonal, alternate direction bracing and eccentric V type bracing.

4. Model 3 has maximum reduction in drift index percentage of 71.757 % in comparison with the un-braced reference model in the X direction

5. The axial loads on the columns increase in their value by 44.343% and 51.157% by using model 4 and model 1 respectively.

6. The column moments have increased by 14.027% and 2.085% by using model 1 and model 4 respectively.

7. The column moments have reduced by 8.789% and 28.898% by using model 6 and model 7.

8. Eccentrically braced type of model seven is more economical compared to other bracings. But due to its lateral displacement is high; it is not efficient to resist lateral loads.

9. Single diagonal alternate direction bracing system (model 3) is both economical and laterally stable; because it's lateral displacement is small compared to others.

10. The lateral storey displacements of the building is greatly reduced by the use of eccentric (V) type bracing in comparison to concentric (X and single diagonal) bracing system.

Therefore; from the four concentric and four eccentric types of bracings, the single diagonal (model 3) and eccentric V type (model 5) is the effective bracing system by reducing lateral displacement. Whereas; by considering lateral stiffness and economy the concentric (single diagonal, alternate direction) bracing is the most suitable one for the steel building studied under the present study.

5.2 Recommendation

This thesis work is an inch towards the complex phenomena to select the performance of various bracing types. The main purpose of this study is that when we compare different things first criteria of comparisons should be set to treats the given conditions equally.

From this study it is possible to recommend that for high rise irregular steel building, single diagonal, alternate direction bracing and eccentric (V) type bracings are the effective bracing systems in comparison to other eccentric and concentric bracing systems. In the case of eccentric, the value of link length “e” should be in the range of ($e = e/L \leq 0.5$) unless the lateral stiffness gained from it is not efficient to resist lateral loads.

5.3. Future Scope of the work

Under this study the sample of the bracing type that I considered is classified under concentrically bracing and eccentrically bracing. Among the possibilities for future study, the following are the main points that deserve attention.

1. In this study it is only considered concentrically and eccentrically type of bracing for resisting lateral loads. A study for steel plate shear wall system to resist lateral loads in comparison to different bracing system is left for future investigation.

2. The analysis takes place by selecting a double channel steel section for bracing cross section. The next researcher is expected to check the structural behavior under another cross-section like angle section, tubular section, etc.

3. In this study aesthetics value of bracing system is not considered for openings (doors, windows) the next researcher is expected to consider in the future study.

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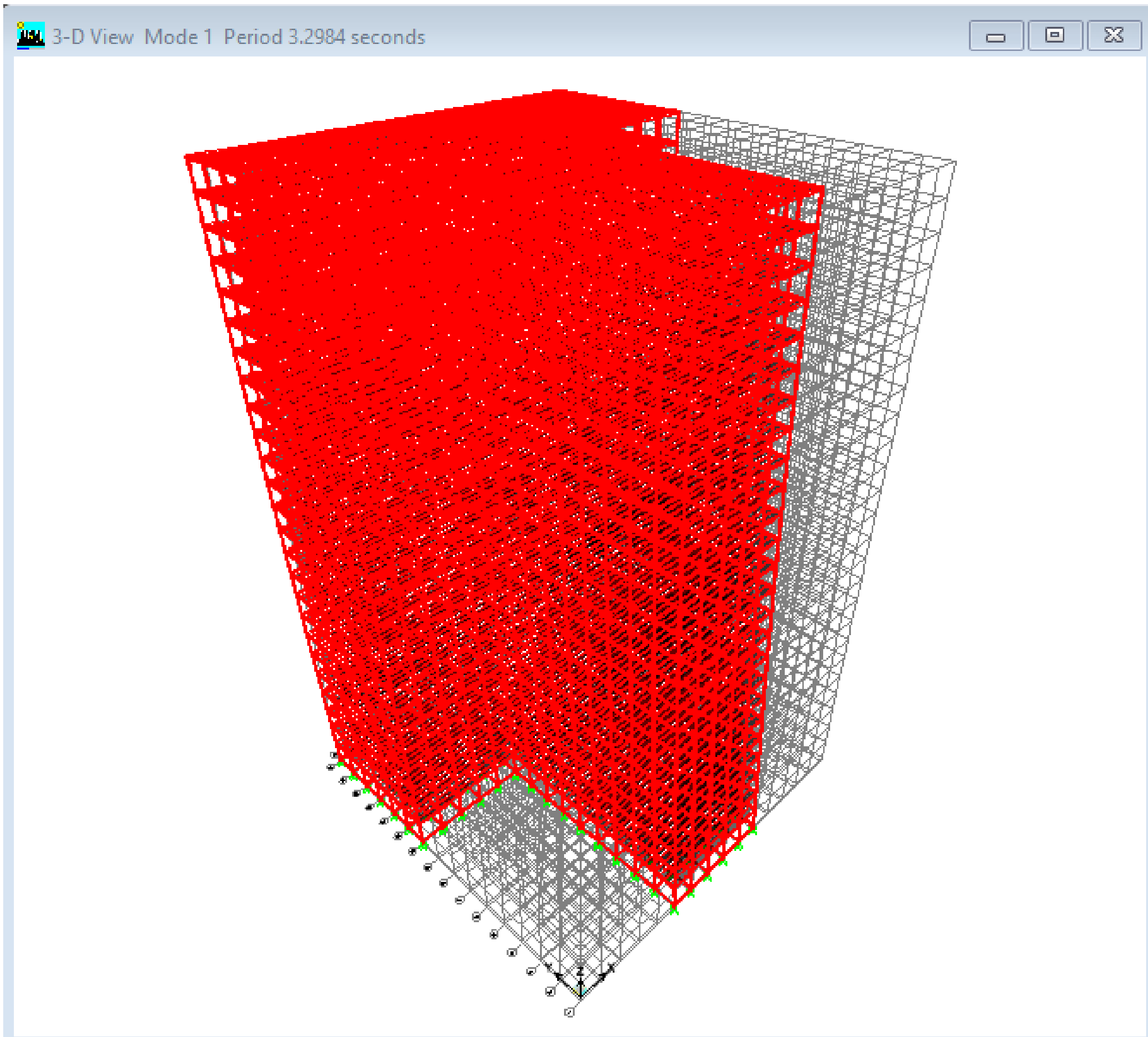
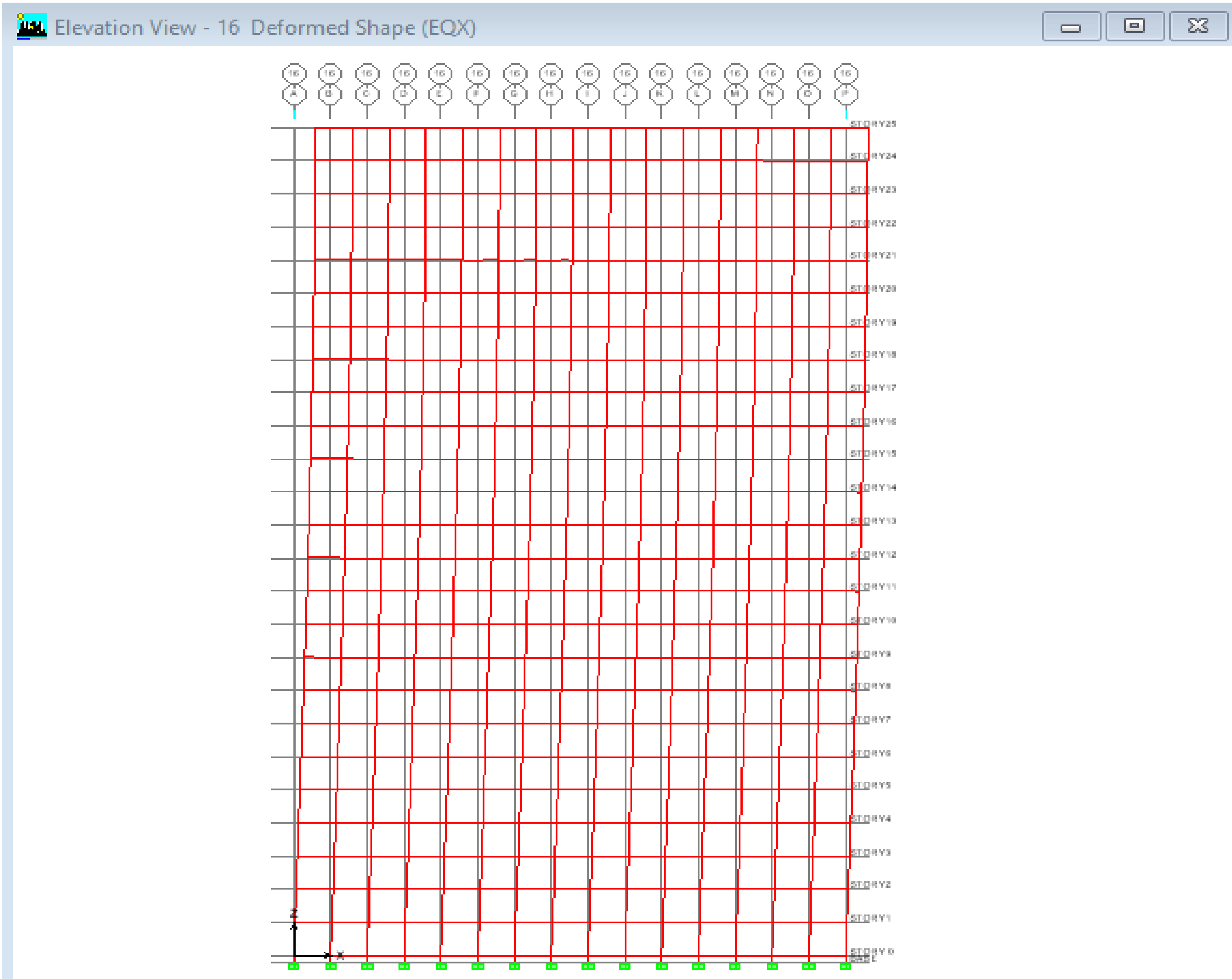
APENDIXES. A1

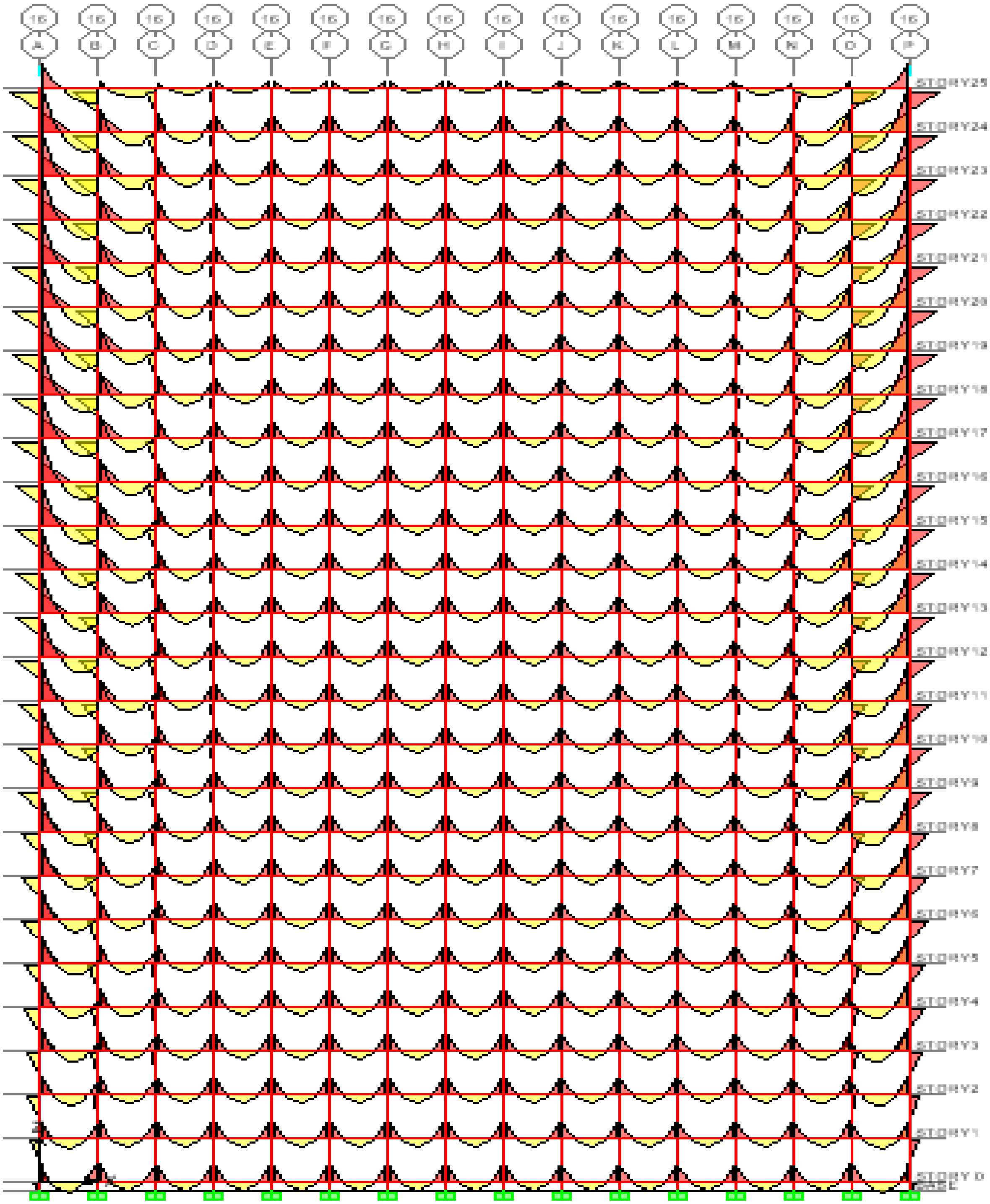
The following out puts are lateral displacement, story drift, axial force diagram and bending moment diagrams of un braced braced, diagonal braced and knee braced G+25 irregular T Plan steel building.



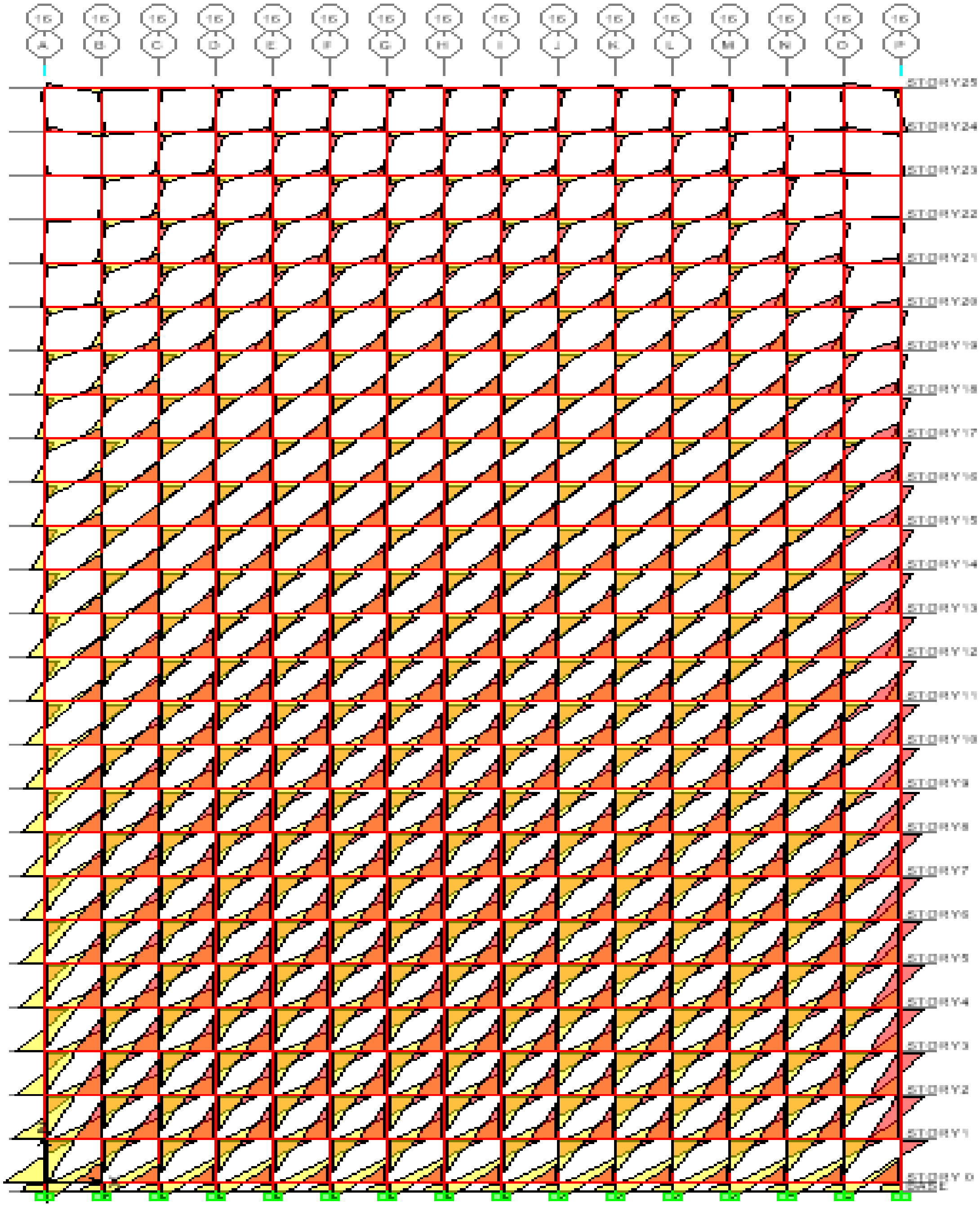
Deformed shape of un braced model



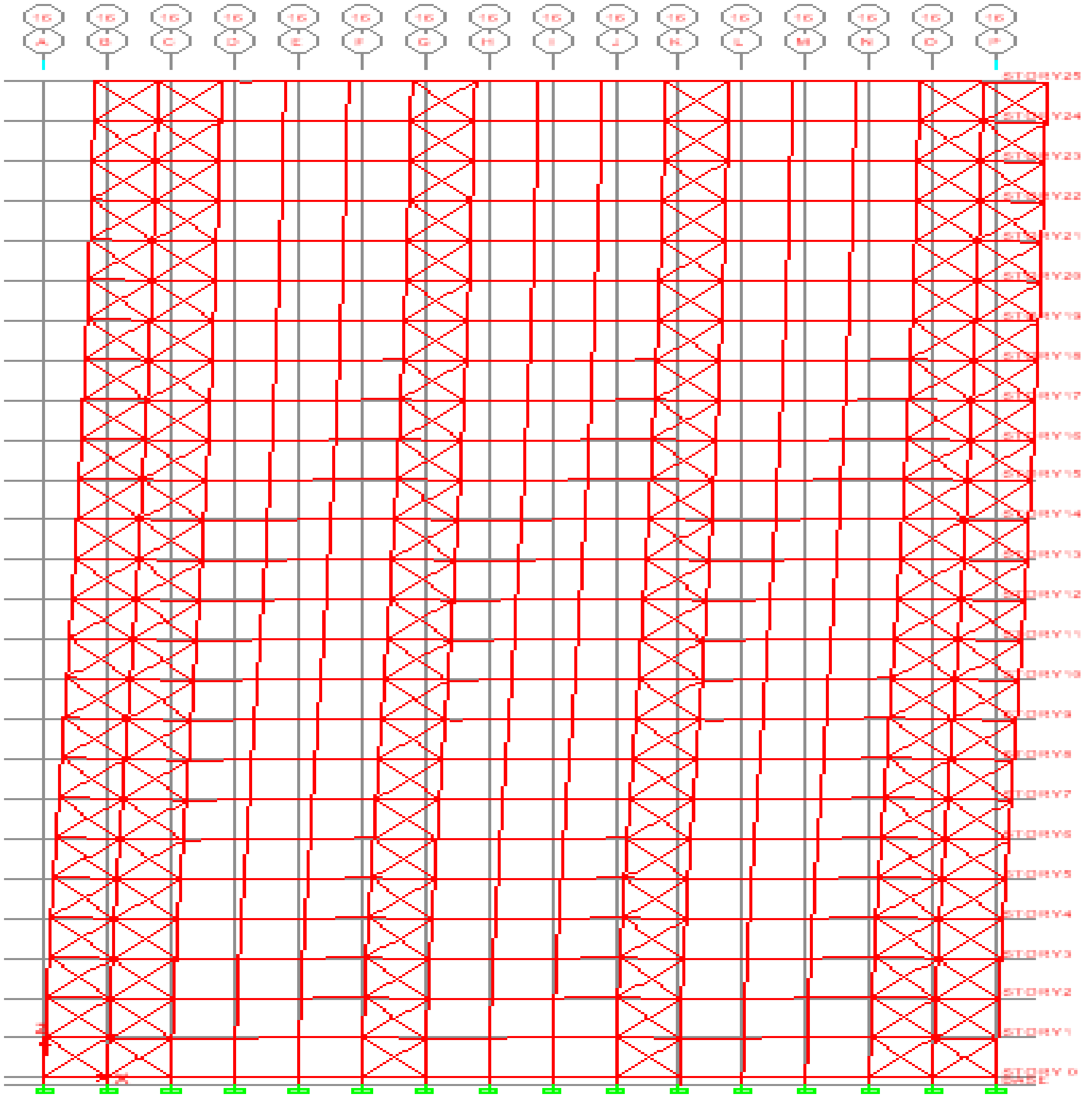
Deformed shape (nodal displacement) of unbraced due to lateral load (EQX)



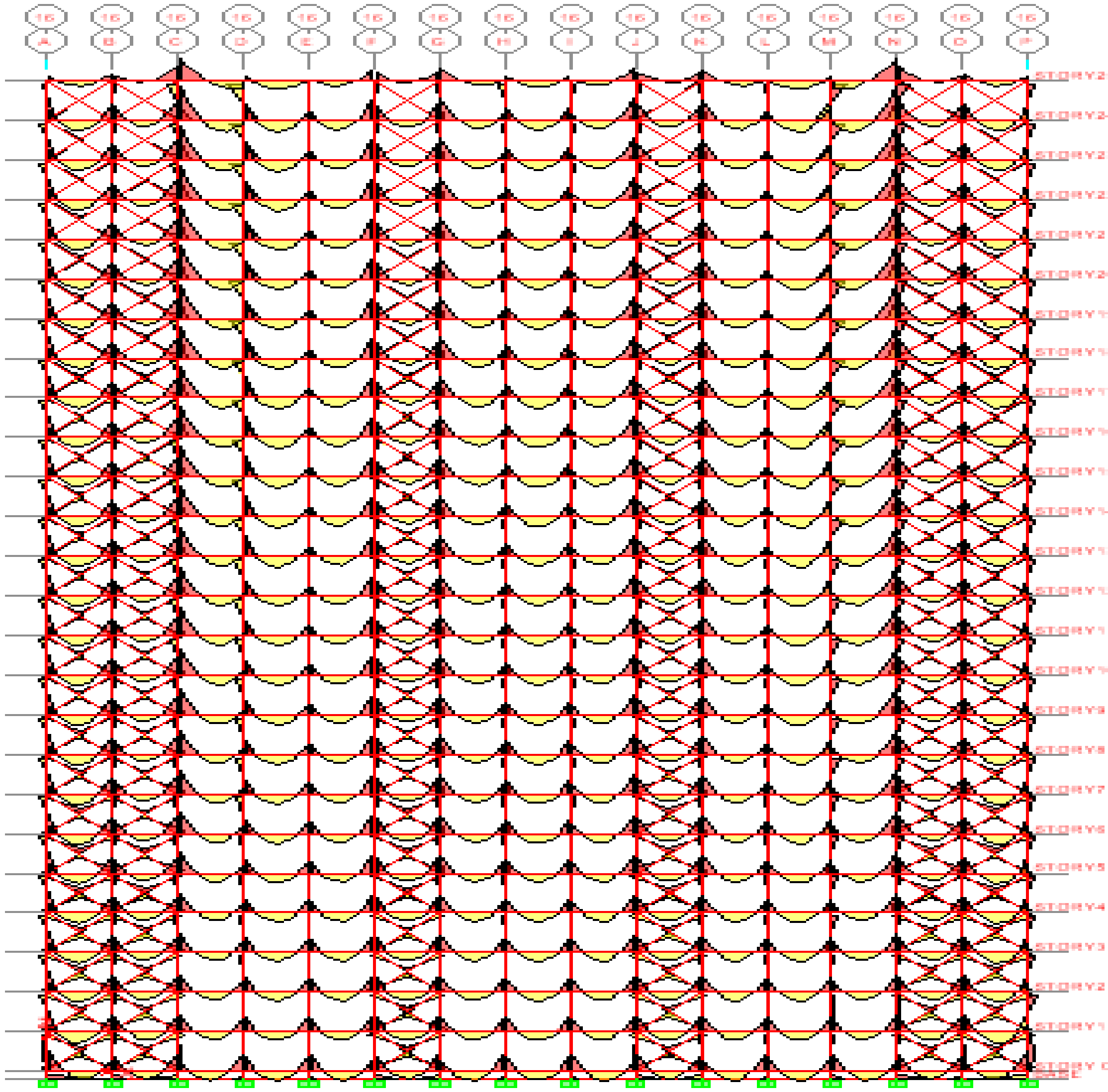
Elevation view 16 moment 3-3 diagram (dead)



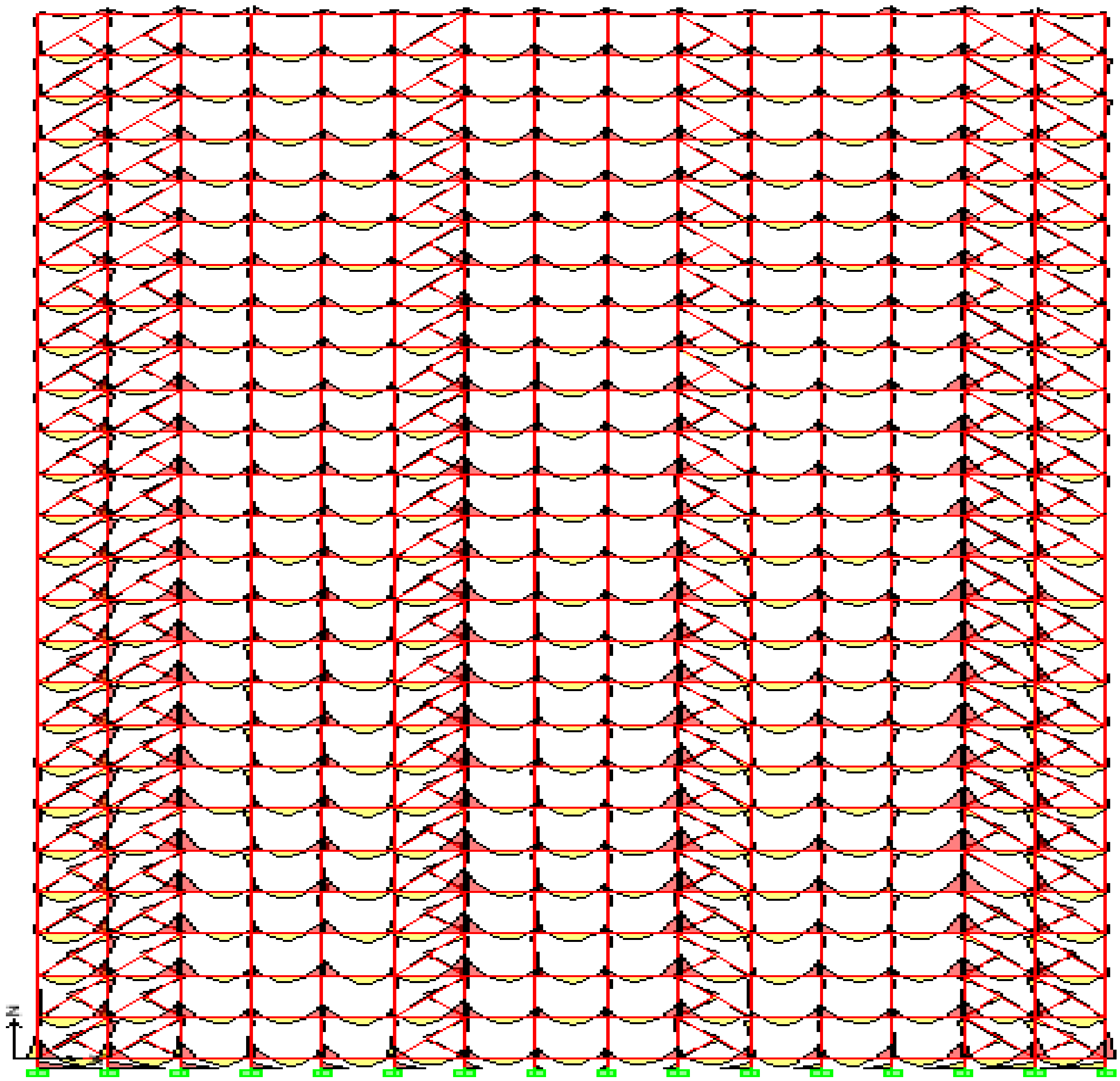
Elevation view 16 moment 3-3 diagram (EQX)



Deformed shape (nodal displacement) of x bracing due to lateral load (EQX)



Bending moment diagram due to comb 1



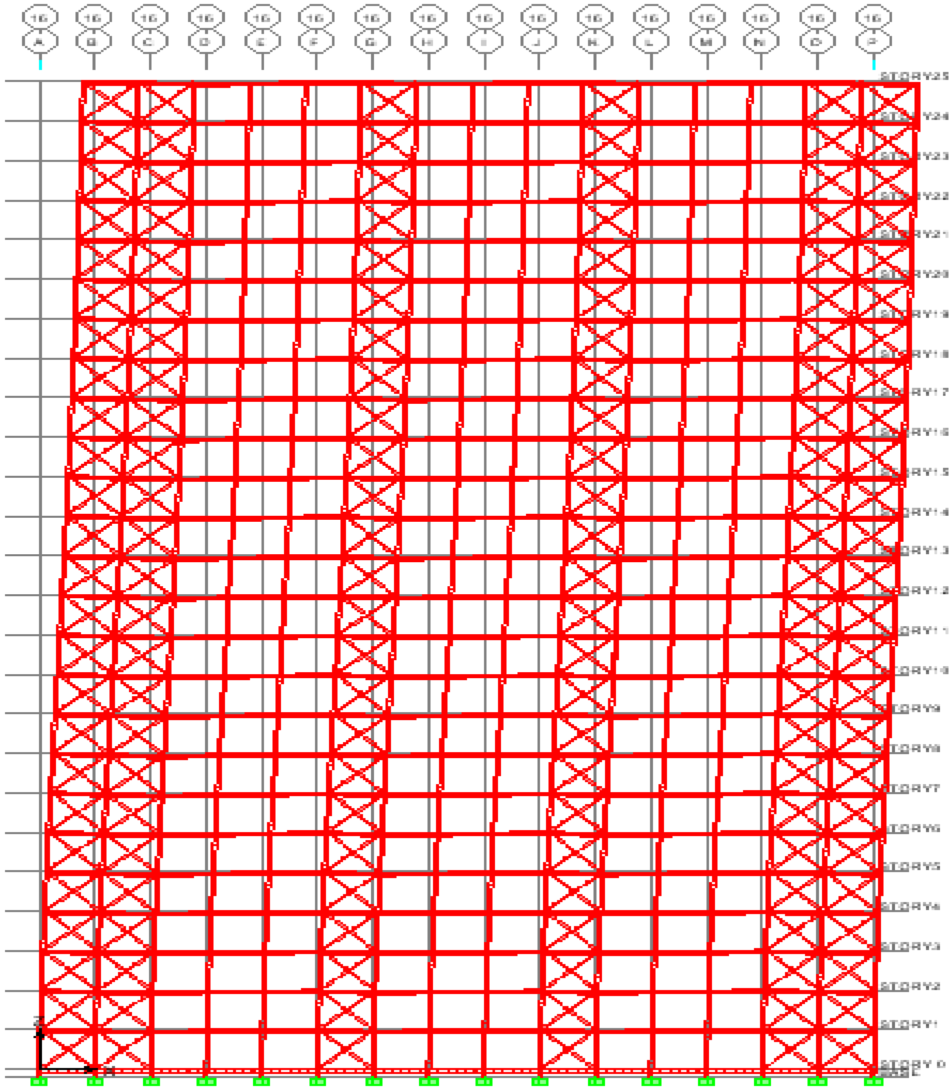
Elevation view, 16 moments 3-3 diagram of knee bracing

DISPLACEMENTS AND DRIFTS AT POINT OBJECT 16

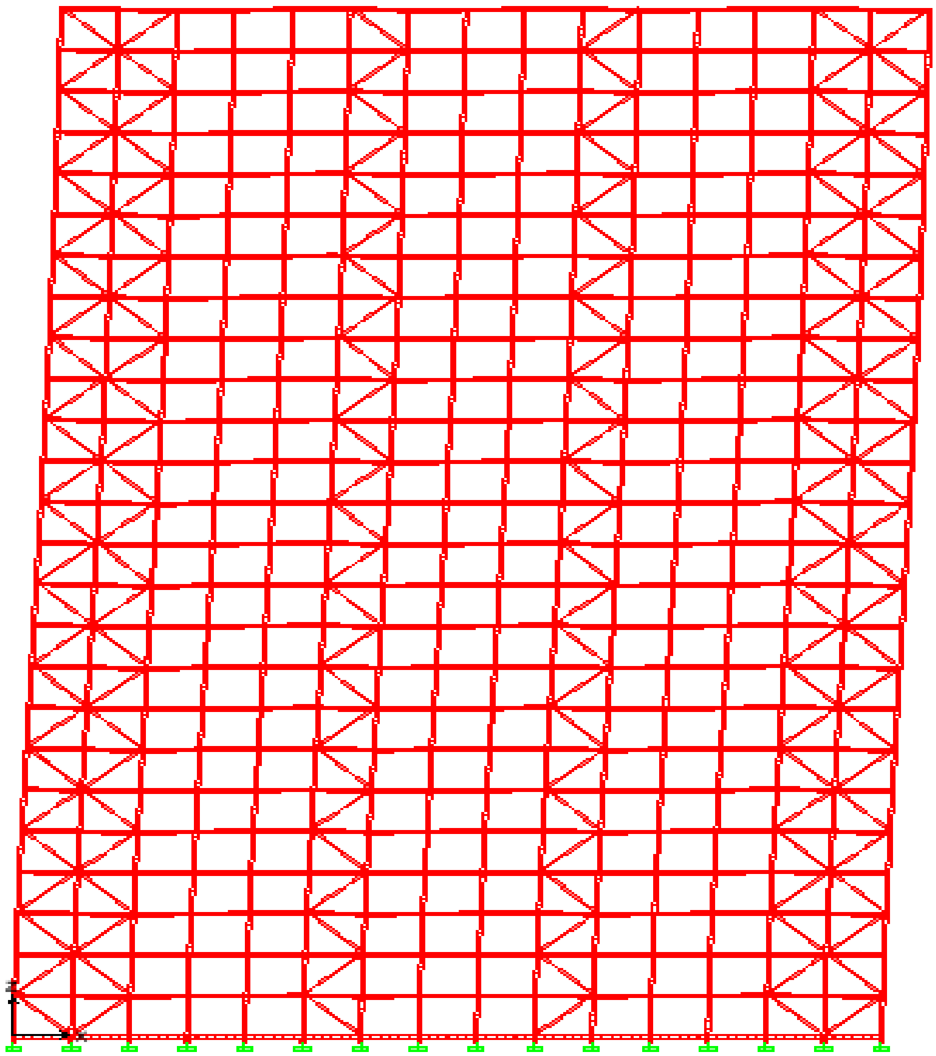
File

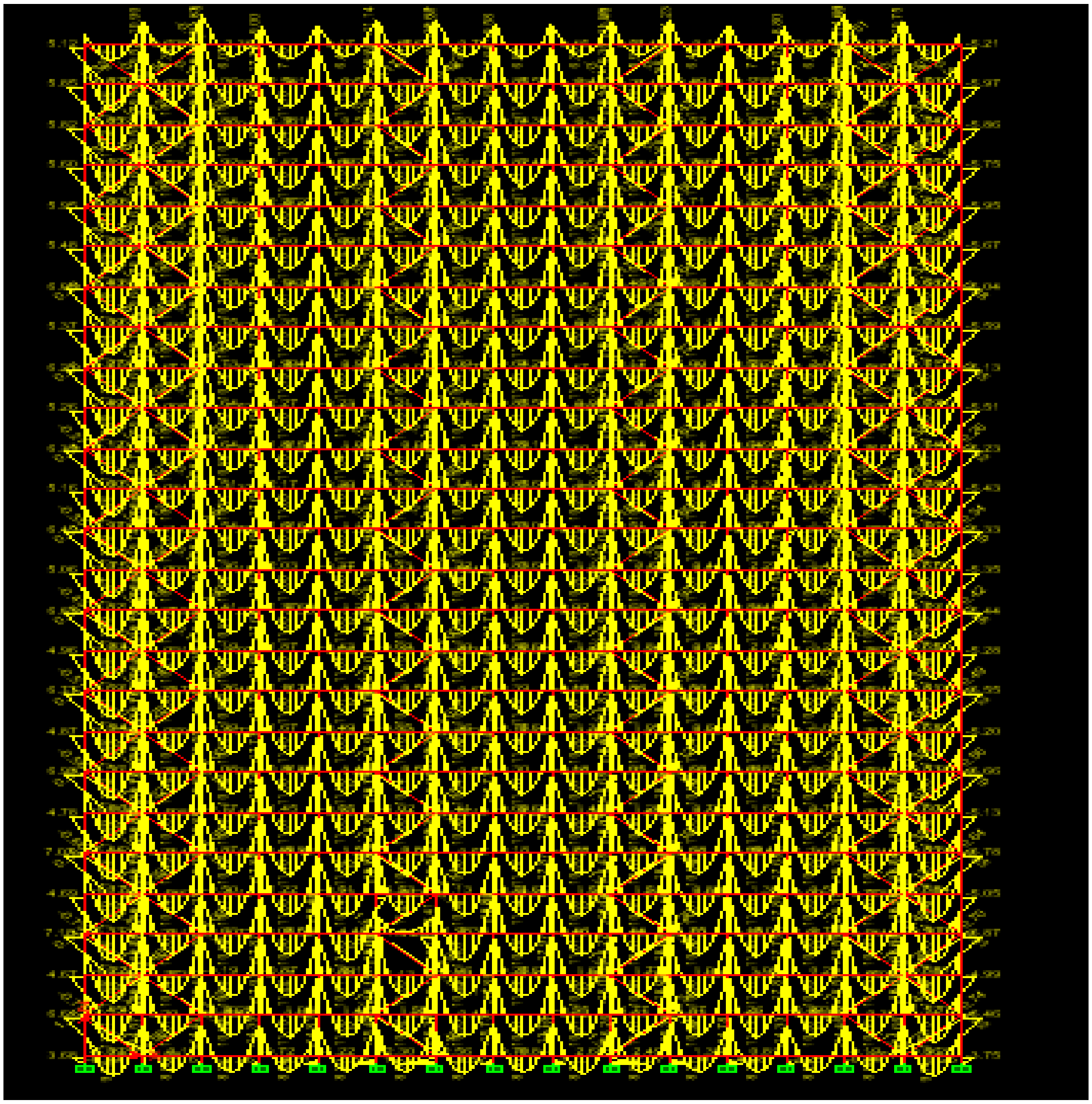
STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
STORY25	10.494953	-0.893519	0.069688	0.002357
STORY24	10.216200	-0.884091	0.071521	0.004729
STORY23	9.930114	-0.865177	0.075454	0.005795
STORY22	9.628296	-0.841996	0.080327	0.006480
STORY21	9.306989	-0.816076	0.085758	0.007037
STORY20	8.963958	-0.787928	0.091546	0.007558
STORY19	8.597774	-0.757696	0.097535	0.008073
STORY18	8.207636	-0.725404	0.103583	0.008587
STORY17	7.793302	-0.691057	0.109554	0.009096
STORY16	7.355087	-0.654675	0.115306	0.009591
STORY15	6.893864	-0.616312	0.120694	0.010062
STORY14	6.411087	-0.576065	0.125564	0.010494
STORY13	5.908831	-0.534088	0.129748	0.010870
STORY12	5.389838	-0.490607	0.133076	0.011178
STORY11	4.857535	-0.445894	0.135346	0.011395
STORY10	4.316150	-0.400316	0.136354	0.011507
STORY9	3.770734	-0.354289	0.135873	0.011499
STORY8	3.227243	-0.308292	0.133652	0.011355
STORY7	2.692633	-0.262874	0.129416	0.011059
STORY6	2.174968	-0.218636	0.122860	0.010602
STORY5	1.683530	-0.176229	0.113634	0.009975
STORY4	1.228993	-0.136328	0.101331	0.009179
STORY3	0.823669	-0.099611	0.085442	0.008197
STORY2	0.481903	-0.066825	0.065127	0.007015
STORY1	0.221397	-0.038763	0.039578	0.005949
STORY 0	0.063084	-0.014966	0.078855	0.018707

Lateral displacement values of x bracing

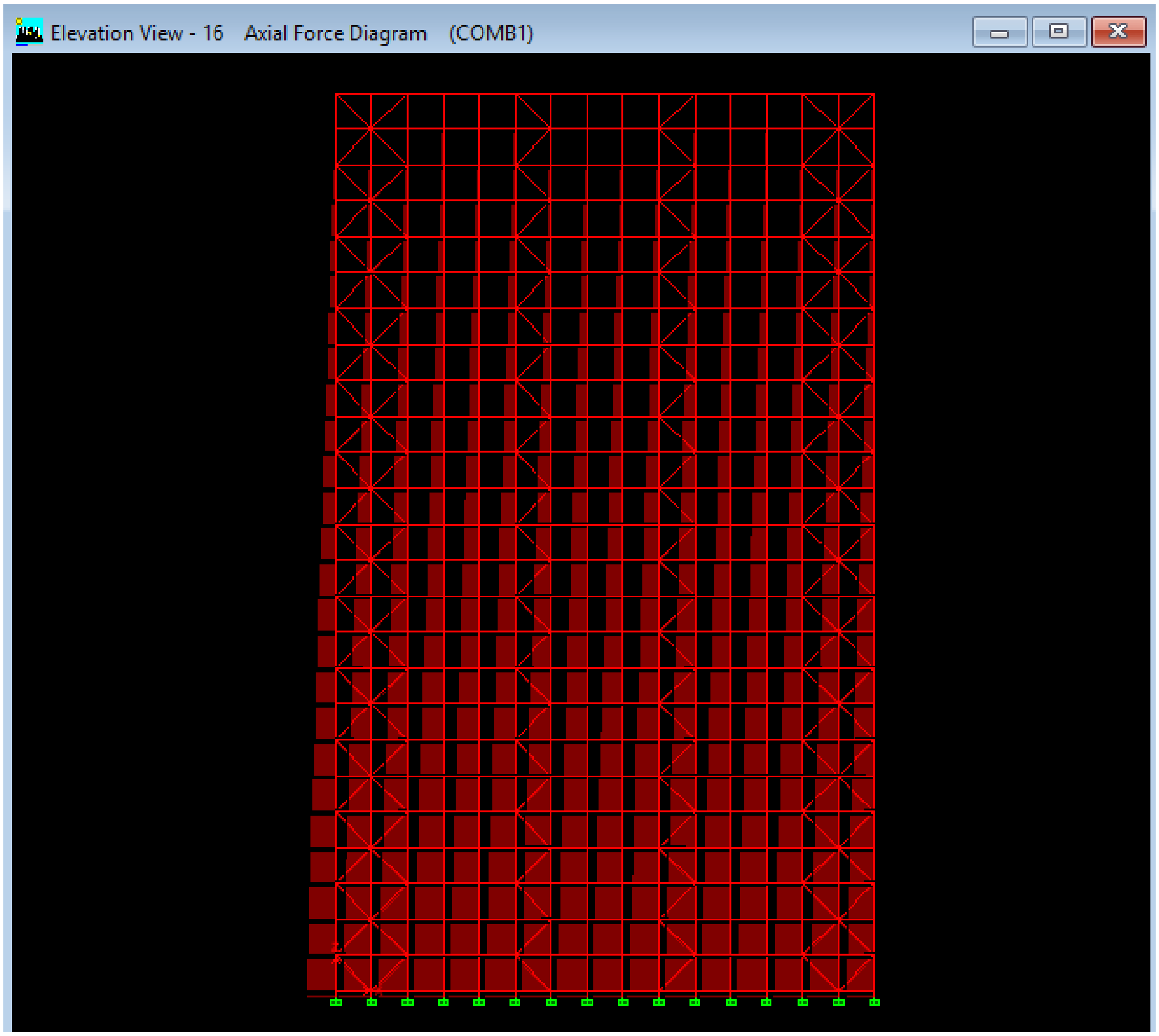


Lateral displacement diagram of x bracing





Bending moment diagram of single diagonal olternate direction bracing



Axial force diagram of single diagonal alternate direction bracing

Story	Point	Load	UX	UY	UZ	RX	RY	RZ
STORY23	156	EQX	5.4781	0.0945	-0.0004	-0.00059	0.01339	0.01426
STORY23	157	EQX	5.4211	0.0945	-0.0010	-0.00027	0.01297	0.01426
STORY23	158	EQX	5.3641	0.0945	-0.0018	0.00063	0.01276	0.01426
STORY23	159	EQX	5.3070	0.0945	0.0140	0.00828	0.01544	0.01426
STORY23	160	EQX	5.2500	0.0945	0.0714	0.01084	0.02413	0.01425
STORY23	161	EQX	6.1053	0.1509	-0.1211	-0.00164	0.04403	0.01420
STORY23	162	EQX	6.0487	0.1509	-0.1306	-0.00249	0.03646	0.01416
STORY23	163	EQX	5.9920	0.1509	-0.1346	0.00406	0.03382	0.01416
STORY23	164	EQX	5.9352	0.1511	-0.0980	0.00359	0.03064	0.01421
STORY23	165	EQX	5.8783	0.1513	-0.0989	0.00176	0.03117	0.01424
STORY23	166	EQX	5.8213	0.1514	-0.0791	0.00099	0.02917	0.01426
STORY23	167	EQX	5.7642	0.1516	-0.0782	0.00849	0.02982	0.01428
STORY23	168	EQX	5.7070	0.1516	-0.0016	0.00935	0.02301	0.01427
STORY23	169	EQX	5.6497	0.1517	0.0006	-0.00158	0.02368	0.01432
STORY23	170	EQX	5.5921	0.1517	-0.0006	-0.00039	0.03176	0.01430
STORY23	171	EQX	5.5351	0.1516	-0.0034	-0.00015	0.01710	0.01426
STORY23	172	EQX	5.4781	0.1516	-0.0006	0.00052	0.01286	0.01425
STORY23	173	EQX	5.4211	0.1515	0.0014	0.00036	0.01282	0.01425
STORY23	174	EQX	5.3640	0.1515	0.0023	-0.00085	0.01273	0.01425
STORY23	175	EQX	5.3070	0.1515	-0.0145	-0.00890	0.01525	0.01426
STORY23	176	EQX	5.2499	0.1515	-0.0751	-0.01167	0.02495	0.01429
STORY23	186	EQX	5.5919	0.2086	-0.1500	0.02249	0.01433	0.01424
STORY23	187	EQX	5.5350	0.2086	-0.0304	0.01739	0.01315	0.01424
STORY23	188	EQX	5.4780	0.2086	0.0035	0.00147	0.01314	0.01424
STORY23	189	EQX	5.4211	0.2085	0.0023	-0.00059	0.01341	0.01425
STORY23	190	EQX	5.3640	0.2085	0.0017	-0.00066	0.01327	0.01425
STORY23	191	EQX	5.3070	0.2085	-0.0075	-0.00454	0.00973	0.01425
STORY23	192	EQX	5.2499	0.2085	-0.0372	-0.00587	-0.00055	0.01424
STORY23	202	EQX	5.5916	0.2656	0.0035	-0.00058	-0.01673	0.01422
STORY23	203	EQX	5.5349	0.2656	0.0020	-0.00056	0.00695	0.01422
STORY23	204	EQX	5.4780	0.2655	-0.0003	-0.00061	0.01428	0.01423
STORY23	205	EQX	5.4211	0.2655	-0.0018	-0.00039	0.01407	0.01424
STORY23	206	EQX	5.3640	0.2654	-0.0001	0.00162	0.01368	0.01425
STORY23	207	EQX	5.3069	0.2654	0.0197	0.00712	0.00904	0.01426

Point displacement values of single diagonal ,alternate bracing

Story	Point	Load	UX	UY	UZ	RX	RY	RZ
STORY25	10	EQX	5.8742	-0.4363	0.1643	0.00402	0.03786	0.01671
STORY25	11	EQX	5.8115	-0.4350	0.1829	0.00502	0.03663	0.01509
STORY25	12	EQX	5.7517	-0.4340	0.1918	-0.00561	0.03514	0.01486
STORY25	13	EQX	5.6926	-0.4332	0.1453	-0.00487	0.02868	0.01470
STORY25	14	EQX	5.6341	-0.4326	0.1589	0.00463	0.03147	0.01463
STORY25	15	EQX	5.5762	-0.4317	0.1685	0.00102	0.03443	0.01438
STORY25	16	EQX	5.5206	-0.4309	0.1708	0.00049	0.03616	0.01317
STORY25	26	EQX	5.8721	-0.3748	-0.0041	0.00918	0.04446	0.01504
STORY25	27	EQX	5.8113	-0.3747	0.0369	0.00819	0.02993	0.01501
STORY25	28	EQX	5.7516	-0.3745	0.0582	0.00069	0.02491	0.01484
STORY25	29	EQX	5.6925	-0.3743	0.0482	-0.00173	0.01975	0.01472
STORY25	30	EQX	5.6340	-0.3741	0.0460	-0.00094	0.02165	0.01461
STORY25	31	EQX	5.5761	-0.3740	0.0337	-0.00425	0.02796	0.01448
STORY25	32	EQX	5.5188	-0.3739	0.0114	-0.00460	0.04138	0.01447
STORY25	42	EQX	5.8713	-0.3153	-0.1609	0.02896	0.00918	0.01515
STORY25	43	EQX	5.8110	-0.3153	-0.0337	0.02134	0.00986	0.01487
STORY25	44	EQX	5.7515	-0.3153	0.0123	0.00344	0.01104	0.01479
STORY25	45	EQX	5.6925	-0.3153	0.0135	-0.00030	0.01021	0.01471
STORY25	46	EQX	5.6339	-0.3154	0.0092	-0.00369	0.00994	0.01464
STORY25	47	EQX	5.5758	-0.3154	-0.0339	-0.01827	0.01122	0.01458
STORY25	48	EQX	5.5183	-0.3154	-0.1402	-0.02355	0.01561	0.01430
STORY25	58	EQX	5.8697	-0.2562	-0.0109	0.00145	-0.03451	0.01461
STORY25	59	EQX	5.8106	-0.2562	-0.0048	0.00137	-0.00211	0.01477
STORY25	60	EQX	5.7514	-0.2563	-0.0001	0.00090	0.00812	0.01475
STORY25	61	EQX	5.6925	-0.2564	0.0024	0.00040	0.00805	0.01471
STORY25	62	EQX	5.6339	-0.2565	0.0002	-0.00209	0.00742	0.01467
STORY25	63	EQX	5.5755	-0.2566	-0.0216	-0.00843	0.00106	0.01465
STORY25	64	EQX	5.5170	-0.2566	-0.0688	-0.01032	-0.01542	0.01479
STORY25	74	EQX	5.8691	-0.1974	0.1458	-0.02668	0.00788	0.01464
STORY25	75	EQX	5.8102	-0.1973	0.0294	-0.01823	0.00630	0.01471
STORY25	76	EQX	5.7512	-0.1974	-0.0039	-0.00143	0.00642	0.01471
STORY25	77	EQX	5.6924	-0.1975	-0.0024	0.00062	0.00694	0.01469
STORY25	78	EQX	5.6338	-0.1976	-0.0018	0.00054	0.00684	0.01468
STORY25	79	EQX	5.5753	-0.1977	0.0071	0.00467	0.00220	0.01468

Point displacement values of single diagonal ,alternate bracing

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY
STORY25	Max Drift X	EQX	161	40.000	0.000	100.000	0.035786	
STORY25	Max Drift Y	EQX	10	0.000	36.000	100.000		0.002588
STORY24	Max Drift X	EQX	145	36.000	0.000	96.000	0.038950	
STORY24	Max Drift Y	EQX	12	0.000	44.000	96.000		0.002035
STORY23	Max Drift X	EQX	81	20.000	0.000	92.000	0.041609	
STORY23	Max Drift Y	EQX	11	0.000	40.000	92.000		0.002318
STORY22	Max Drift X	EQX	145	36.000	0.000	88.000	0.045261	
STORY22	Max Drift Y	EQX	16	0.000	60.000	88.000		0.002608
STORY21	Max Drift X	EQX	81	20.000	0.000	84.000	0.048235	
STORY21	Max Drift Y	EQX	11	0.000	40.000	84.000		0.002870
STORY20	Max Drift X	EQX	145	36.000	0.000	80.000	0.052542	
STORY20	Max Drift Y	EQX	15	0.000	56.000	80.000		0.003191
STORY19	Max Drift X	EQX	81	20.000	0.000	76.000	0.055573	
STORY19	Max Drift Y	EQX	11	0.000	40.000	76.000		0.003453
STORY18	Max Drift X	EQX	145	36.000	0.000	72.000	0.060189	
STORY18	Max Drift Y	EQX	15	0.000	56.000	72.000		0.003794
STORY17	Max Drift X	EQX	81	20.000	0.000	68.000	0.063063	
STORY17	Max Drift Y	EQX	251	60.000	40.000	68.000		0.004094
STORY16	Max Drift X	EQX	145	36.000	0.000	64.000	0.067653	
STORY16	Max Drift Y	EQX	15	0.000	56.000	64.000		0.004395
STORY15	Max Drift X	EQX	81	20.000	0.000	60.000	0.070163	
STORY15	Max Drift Y	EQX	251	60.000	40.000	60.000		0.004715
STORY14	Max Drift X	EQX	145	36.000	0.000	56.000	0.074362	
STORY14	Max Drift Y	EQX	250	60.000	36.000	56.000		0.005130
STORY13	Max Drift X	EQX	81	20.000	0.000	52.000	0.076279	
STORY13	Max Drift Y	EQX	256	60.000	60.000	52.000		0.005457
STORY12	Max Drift X	EQX	145	36.000	0.000	48.000	0.079664	
STORY12	Max Drift Y	EQX	250	60.000	36.000	48.000		0.005947
STORY11	Max Drift X	EQX	81	20.000	0.000	44.000	0.080713	
STORY11	Max Drift Y	EQX	256	60.000	60.000	44.000		0.006179
STORY10	Max Drift X	EQX	145	36.000	0.000	40.000	0.082780	
STORY10	Max Drift Y	EQX	250	60.000	36.000	40.000		0.006682
STORY9	Max Drift X	EQX	113	28.000	0.000	36.000	0.082637	
STORY9	Max Drift Y	EQX	256	60.000	60.000	36.000		0.006767

Point displacement values of single diagonal ,alternate bracing